Shri Vithal Education & Research Institute's

COLLEGE OF ENGINEERING, PANDHARPUR

P.B.No.54, Gopalpur - Ranjani Road, Gopalpur, Pandharpur - 413304, District. Solapur (Maharashtra)
Tel.: (02186) 216063, 9503103757, Toll Free No.: 1800-3000-4131 e-mail.: coe@sveri.ac.in
Website.: www.sveri.ac.in (Approved by A.I.C.T.E., New Delhi and Affiliated to Solapur University, Solapur)
NBA Accredited all eligible UG Programmes, NAAC Accreditated Institute,ISO 9001:2015 Certified Institute.
Accredited by The Institution of Engineers (India), Kolkata and TCS, Pune.

SVERI
Engineering for Excellence

Ref.:- Date:-

1.3.3 Number of the student studied course on experimental learning through Project Work / Internship

Programme Name: M.Tech. Civil -Structural Engineering Programme Code: 1-1408968343			
Year of offering: 2019-2020			
Sr. No.	Name of the Course that include experiential learning through project work/field work/internship	Course code	Number of the student studied course on experiential learning through project work/field work/internship
1.	Mini project		18
2.	Dissertation Phase I : Synopsis Submission Seminar		13
3.	Dissertation Phase II : ICA		
4.	Dissertation Phase II : Progress Seminar		
5.	Dissertation Phase III : Progress Seminar		
6.	Dissertation Phase IV : Final presentation and submission of report		
7.	Dissertation and Viva-voce		

COLLEGE OF ENGINEERING PANDHARPUR

B. Range

PRINCIPAL,
College of Engineering
PANDHARPUR

O/C

Shri Vithal Education & Research Institute's

COLLEGE OF ENGINEERING, PANDHARPUR

P.B.No.54, Gopalpur - Ranjani Road, Gopalpur, Pandharpur - 413304, **District**: Solapur (Maharashtra)
Tel.: (02186) 216063, 9503103757, **Toll Free No.**: 1800-3000-4131 **e-mail.**: coe@sveri.ac.in
Website.: www.sveri.ac.in (Approved by A.I.C.T.E., New Delhi and Affiliated to Solapur University, Solapur)
NBA Accredited all eligible UG Programmes, **NAAC** Accreditated Institute,ISO 9001:2015 Certified Institute.
Accredited by The Institution of Engineers (India), Kolkata and TCS, Pune.

Ref.:- COEPR/2019-20/563 Date:- 28/08/2019

To
Hon. Pro-Vice Chancellor,
Punyashlok Ahilyadevi Holkar Solapur University,
Dnyanteerth Nagar, Kegaon,
Solapur-Pune National Highway, Solapur- 413255.

Sub.:- Regarding submission of Synopsis of M.Tech. Dissertation of M.Tech. Civil (Structures) students.

Respected Sir,

We are submitting herewith Synopsis of our 09 M. Tech. Civil (Structures) students as per the details given in Annexure-I, attached herewith.

You are requested to kindly take up the Synopsis for further consideration and approval of the same.

Thanking you,

Yours faithfully,

B. Ronge

(Dr. B. P. Ronge)
PRINCIPAL

Encl:

1. List of students along with Titles of Synopsis of M.Tech Dissertation.
2. Six copies of Synopsis of each student.

Received
29/08/19
लिपिक
आवक - जावक विभाग
शैक्षणिक, संशोधन व विकास
सोलापूर विद्यापीठ, सोलापूर.

COLLEGE OF ENGINEERING
PANDHARPUR

SVERI's College of Engineering, Pandharpur
Department of Civil Engineering.

Annexure-I

M. Tech. Civil (Structures)

Year 2019-20

Sr. No.	Name of Candidate	Title of Synopsis	Year of Registration	Name of Guide
1	Abhangrao Chaitali Rajendra	"Analysis of Cement Performance Using Microscopic Analysis Techniques"	2018	Dr. Prashant M. Pawar
2	Bagal Srushti Chandrakant	"Performance Based Seismic Design Analysis of Steel Frames."	2018	Dr. Prashant M. Pawar
3	Deshmane Shweta Anil	"Evaluation of Steel Fiber Reinforced Alkali Activated Slag Concrete along with Fly Ash."	2018	Dr. Ms. V. S. Kshirsagar
4	Ingale Sujata Janardhan	"Ultrasonic Pulse Velocity Performance Analysis of Concrete Under Various Conditions."	2018	Dr. Prashant M. Pawar
5	Jadhav Surekha Mahadev	"Numerical Analysis of Hollow Circular Monopile Structure."	2018	Dr. Prashant M. Pawar
6	Lachyan Pooja Shivanand	"Evaluation of Lateral Stability of Cantilever Rataining Wall."	2018	Prof. S. A. Gosavi
7	Mahamuni Nishigandha Vaijinath	"Design and Analysis of Slabs with Insertion Voids for Weight Minimization."	2018	Dr. Prashant M. Pawar
8	Patil Swapnil Mohan	"Analytical evaluation of seismic response reduction factor for steel frame building by using various types of steel	2018	Dr. Prashant M. Pawar
9	Pawar Prasad Shivaji	"Comparative Evaluation of Strength Properties of Green Concrete and Conventional Concrete."	2018	Prof. M. M. Pawar

COLLEGE OF ENGINEERING * PANDH...

B.Ronge
(Dr. B. P. Ronge)
PRINCIPAL

SOLAPUR UNIVERSITY

Punyashlok Ahilyadevi Holkar
Solapur University Solapur

Receipt (F.Y. : 2019 - 2020)

सोलापूर विद्यापीठ
॥ विद्यया संपन्नता ॥

Number	: **9563**	Dated	: **29-Aug-2019**
Received From	: **Prin.College of Engineering Pandharpur**	Exam	:
Address	:	Month/Year	:
Centre Name	:	Seat No	:

Particulars	Amount
M.E. Synopsis Approval Fees	**9,000.00**

Remarks : M.tech Synopsys

सोलापूर विद्यापीठ
॥ विद्यया संपन्नता ॥

INR Nine Thousand Only.

Grand Total ₹ 9,000.00

Clerk

Any discrepancy in the above receipt may kindly be brought to the notice of Cashier within two days.

SHRI VITHAL EDUCATION & RESEARCH INSTITUTE'S

COLLEGE OF ENGINEERING, PANDHARPUR.

Department of Civil Engineering

Acad Year:2019-20 **ME (2nd Year) PROJECTS**

Sr. No.	NAME OF STUDENT	Project Title	NAME OF GUIDE
1	Abhangrao Chaitali Rajendra	"Analysis of Cement Performance Using Microscopic Analysis Techniques"	Dr. Prashant M. Pawar
2	Bagal Srushti Chandrakant	"Performance Based Seismic Design Analysis of Steel Frames."	Dr. Prashant M. Pawar
3	Deshmane Shweta Anil	"Evaluation of Steel Fiber Reinforced Alkali Activated Slag Concrete along with Fly Ash."	Dr. Ms. V. S. Kshirsagar
4	Ingale Sujata Janardhan	"Ultrasonic Pulse Velocity Performance Analysis of Concrete Under Various Conditions."	Dr. Prashant M. Pawar
5	Jadhav Surekha Mahadev	"Numerical Analysis of Hollow Circular Monopile Structure."	Dr. Prashant M. Pawar
6	Lachyan Pooja Shivanand	"Evaluation of Lateral Stability of Cantilever Rataining Wall."	Prof. S. A. Gosavi
7	Mahamuni Nishigandha Vaijinath	"Design and Analysis of Slabs with Insertion Voids for Weight Minimization."	Dr. Prashant M. Pawar
8	Patil Swapnil Mohan	"Analytical evaluation of seismic response reduction factor for steel frame building by using various types of steel bracing"	Dr. Prashant M. Pawar
9	Pawar Prasad Shivaji	"Comparative Evaluation of Strength Properties of Green Concrete and Conventional Concrete."	Prof. M. M. Pawar
10	Mr. Yogesh Gururaj Mane	"Experimental Study On Flexure And Compressive Strength Of The Reinforced Concrete Using Crushed Sand and Steel Fiber."	Prof. S. A. Gosavi
11	MR.SHUBHAM RAJABHAU GADE	"Structural Behaviour Of Voided Beam"	Prof. M .M. Pawar
12	MR. DANDGE PRATAP DHANANJAY	"To Study Performance of Rcc Multistoreyed Building During Progressive Collapse Using Staad-Pro"	Prof. S. A. Gosavi

(S.A. GOSAVI)
Project Co-ordinator

(Dr. P.M. Pawar)
H.O.D. Civil Dept.

SHRI VITHAL EDUCATION & RESEARCH INSTITUBE'S

COLLEGE OF ENGINEERING, PANDHARPUR.

Department of Civil Engineering

Dessertation Phase-I: Synopsis Submission Seminar

Acad Year:2019-20 Sem-III Class-ME (2nd Year)

Sr. No.	Seat No	Name of Student	Disertation Phase I: Synopsis Submission Seminar
1	825139	BAGAL SRUSHTI CHANDRAKANT	48
2	825140	JADHAV SUREKHA MAHADEV	48
3	825141	INGALE SUJATA JANARDHAN	48
4	825144	ABHANGRAO CHAITALI RAJENDRA	48
5	825146	MANE YOGESH GURURAJ	44
6	825147	LACHYAN POOJA SHIVANAND	48
7	825148	MAHAMUNI NISHIGANDHA VAIJINATH	48
8	825151	SHAHABADE VIKRANT BABURAO	42
9	825152	PATIL SWAPNIL MOHAN	48
10	825153	PAWAR PRASAD SHIVAJI	44
11	825155	GADE SHUBHAM RAJABHAU	45
12	825157	DESHMANE SHWETA ANIL	47
13	825158	AIWALE HARI DHARMA	40

(Dr. P.M. Pawar)
H.O.D. Civil Dept.

HEAD,
Dept. of Civil. Engg.
C.O.E. Pandharpur

Scanned with CamScanner

पुण्यश्लोक अहिल्यादेवी होळकर सोलापूर विद्यापीठ, सोलापूर
Punyashlok Ahilyadevi Holkar Solapur University, Solapur
केगाव, सोलापूर - ४१३ २५५, महाराष्ट्र (भारत)
दुरध्वनी : ०२१७-२७४४७७१/७२/७३/ (११ लाईन्स), फॅक्स : ०२१७-२३५१३०
संकेतस्थळ: http://su.digitaluniversity.ac/www.sus.ac.in ई-मेल: bcudpgbutr@sus.ac.in

पुण्यश्लोक अहिल्यादेवी होळकर सोलापूर विद्यापीठ
॥ विद्यया संपन्नता ॥
NAAC Accredited-2015
'B' Grade (CGPA 2.62)

Ph.D Research Section विस्तारीत क्रमांक - १२३, १२४, १२५

Ref No. PAHSUS/ARD/Ph.D.-I/2020/ 679 **Date:** 20 JAN 2020

To,
The Principal,
SVERI's College of Engineering, Pandharpur,
Tal-Pandharpur, Dist-Solapur-413304.

Subject :- Approval of M.E./M.Tech. Civil Engineering Dissertation Title.

Reference :- RRC Meeting Dated 13/01/2020.

Sir/Madam,

With reference to above Subject, I am directed to inform you that, Research & Recognition Committee has accorded approval to the title of **M.E./M.Tech. Civil Engineering** Dissertation, as mentioned overleaf.

You are requested to bring the approval to the notice of concerned guide and students.

Thanking you.

Yours Faithfully,

(Assistant Registrar)
Research Development (Ph.D.-I)

Copy to :-
The Director,
Board of Examinations and Evaluation,
P.A.H., Solapur University, Solapur.

College of Engineering
Pandharpur.
Inward No. 1234
Date 23/01/2020

To, HOD Civil Engg.

P.A.H. SOLAPUR UNIVERSITY, SOLAPUR

Statement showing who have applied for M.E./M.Tech. Dissertation in subject of : **Civil Engineering** **RRC Date: 13/01/2020**

Name of Students and Address	Batch	Name of Guide and Address	Topic of Research work	Recommendations of RRC (with reason)
Miss. Chaitali R. Abhangrao SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	M. Tech 2018	**Dr. P. M. Pawar** SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	Analysis of Cement Performance Using Microscopic Analysis Techniques	**Approved**
Miss. Srushti C. Bagal SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	M. Tech 2018	**Dr. P. M. Pawar** SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	Performance Based Seismic Design Analysis of Steel Frames	**Approved**
Miss. Shweta A. Deshmane SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	M. Tech 2018	Dr. Ms. V. S. Kshirsagar SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	Evaluation of Steel Fiber Reinforced Alkali activated Slag Concrete along with Fly Ash.	**Approved**
Miss. Sujata J. Ingale SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	M. Tech 2018	**Dr. P. M. Pawar** SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	Ultrasonic Pulse Velocity Performance Analysis of Concrete Under Various Conditions	**Approved**
Miss. Surekha M. Jadhav SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	M. Tech 2018	**Dr. P. M. Pawar** SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	Numerical Analysis of Hollow Circular Monopile Structure.	**Approved**
Miss. Pooja S. Lachyan SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	M. Tech 2018	**Prof. S. A. Gosavi** SVERI's College of Engineering, Pandharpur,Tal-Pandharpur, Dist-Solapur-413304	Evaluation of Lateral Stability of Cantilever Walls	**Approved with title corrections**

Name of Students and Address	Batch	Name of Guide and Address	Topic of Research work	Recommendations of RRC (with reason)
Miss. Nishigandha V. Mahamuni SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	M. Tech 2018	**Dr. P. M. Pawar** SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	Design and Analysis of Slabs With Insertion Voids for Weight Minimization	**Approved**
Mr. Swapnil M. Patil SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	M. Tech 2018	**Dr. P. M. Pawar** SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	Evaluation of seismic response reduction factor for steel frame building using various types of steel bracings	**Approved with title corrections**
Mr. Prasad S. Pawar SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	M. Tech 2018	**Prof. M. M. Pawar** SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	Comparative Study of Strength Properties of Green Concrete and Conventional Concrete.	**Approved with title corrections**
Mr. Yogesh Gururaj Mane SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	M. Tech 2018	**Prof. S. A. Gosavi** SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	Experimental Study on Flexure and Compressive Strength of The Reinforced Concrete Using Crushed Sand and Steel Fiber	**Approved**
Mr. Shubham R. Gade SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	M. Tech 2018	**Prof. M. M. Pawar** SVERI's College of Engineering, Pandharpur, Tal-Pandharpur, Dist-Solapur-413304	Structural Behaviour of Voided Beam	**Approved**
Mr. Dandge Pratap Dhananjay SVERI's College of Engneering, Tal- Pandharpur, Dist-Solapur	Aug 2015	**Prof. S.A. Gosavi** SVERI's College of Engneering, Tal- Pandharpur, Dist-Solapur	Study of Performance of RCC Multistoreyed Building During Progessive Collapse	**Approved with title corrections**

शिवाजी ... ०२/०७/१९

(Assistant Registrar)
Research Department (Ph.D.-I)

पुण्यश्लोक अहिल्यादेवी होळकर सोलापूर विद्यापीठ, सोलापूर
Punyashlok Ahilyadevi Holkar Solapur University, Solapur
केगाव, सोलापूर - ४१३ २५५, महाराष्ट्र (भारत)
दुरध्वनी : ०२१७-२७४४७७१/७२/७३/ (११ लाईन्स), फॅक्स : ०२१७-२३५१३०
संकेतस्थळ: http://su.digitaluniversity.ac/www.sus.ac.in ई-मेल: phd@sus.ac.in

NAAC Accredited-2015
'B' Grade (CGPA 2.62)

Ph.D Research Section विस्तारीत क्रमांक - १२३ , १२४, १२५

Ref No. PAHSUS/ARD/Ph.D.-I/2020/ 9993 **Date:** 23 DEC 2020

To,
The Principal,
SVERI's College of Engineering, Pandharpur,
Tal-Pandharpur, Dist-Solapur-413304.

Subject :- Approval of M.E./M.Tech. Civil Engineering Dissertation Title.
Reference :- RRC Meeting Dated 05/11/2020

Sir/Madam,

With reference to above Subject, I am directed to inform you that, Research & Recognition Committee has accorded approval to the title of **M.E./M.Tech. Civil Engineering** Dissertation, as mentioned overleaf.

You are requested to bring the approval to the notice of concerned guide and students.

Thanking you.

Yours Faithfully,

(Deputy Registrar)
Research Department (Ph.D.-I)

Copy to :-
The Director,
Board of Examinations and Evaluation,
P.A.H., Solapur University, Solapur.

P.T.O.

P.A.H.SOLAPUR UNIVERSITY, SOLAPUR

Statement showing who have applied for M.E. Dissertation in subject of : **M.E. Civil Engineering** **Date : 05/11/2020**

Sr. No.	Name of Students and Address	Batch	Name of Guide and Address	Topic of Research work	Recommendations of RRC (with reason)
1.	**Mr. Dhiraj Vitthal Giribuva** SVERI'S College of Engineering, Pandharpur, Solapur.	Aug-2015	**Dr. M. M. Pawar** SVERI'S College of Engineering, Pandharpur.	An Experimental Study of Properties of Concrete with Partial or Full Replacement of Fine Aggregates Through Copper Slag.	**Approved**
2.	**Miss. Mohite Arti Audumbar** SVERI's College of Engineering, Pandharpur, Solapur.	Aug-2019	**Dr. P.M. Pawar** SVERI's College of Engineering, Pandharpur, Solapur.	Performance Based Seismic Design Analysis of Steel Frames for Different types of Bracings	**Approved with Modified title "Performance Based Seismic Design Analysis of Steel Frames for Various types of Bracings"**
3.	**Miss. Umbare Jayshree Ramchandra** SVERI's College of Engineering, Pandharpur, Solapur.	Aug-2019	**Dr. P.M. Pawar** SVERI's College of Engineering, Pandharpur, Solapur.	Progessive Collapse Analysis of Moment Resisting Steel Frames for Failure Performance Improvement	**Approved**
4.	**Mr. Maske Sameer Suhas** SVERI's College of Engineering, Pandharpur, Solapur.	Aug-2019	**Prof. M. M. Pawar** SVERI's College of Engineering, Pandharpur, Solapur.	Assessment of Response Reduction Factor for High Rise Irregular Structures	**Approved**
5.	**Mr. Jadhav Ravikiran Pandurang** SVERI's College of Engineering, Pandharpur, Solapur.	Aug-2019	**Dr. P.M. Pawar** SVERI's College of Engineering, Pandharpur, Solapur.	Analysis of Components of Frame for Improving the Integrity Performance	**Approved**
6.	**Mr. Waghmare Rohit Hanamant** SVERI's College of Engineering, Pandharpur, Solapur.	Aug-2019	**Dr. S.A. Gosavi** SVERI's College of Engineering, Pandharpur, Solapur.	Analysis of box culvert with non-prismatic members for various aspect ratio of cells	**Approved with Modified title "Analysis of box culvert with non-prismatic members for various aspect ratio of cells"**

Statement showing who have applied for M.E. Dissertation in subject of : **M.E. Civil Engineering** **Date : 05/11/2020**

Sr. No.	Name of Students and Address	Batch	Name of Guide and Address	Topic of Research work	Recommendations of RRC (with reason)
7.	**Mr. Kamble Vishwanath Tukaram** SVERI's College of Engineering, Pandharpur, Solapur.	Aug-2019	**Dr. V. S. Kshirsagar** SVERI's College of Engineering, Pandharpur.	Analysis of Seismic Performance of Structure with Floating Column	**Approved with Modified title** **"Seismic Performance of Structures with Floating Columns"**
8.	**Miss. Patil Swati Dattatray** SVERI's College of Engineering, Pandharpur, Solapur.	Aug-2019	**Dr. S. A. Gosavi** SVERI's College of Engineering, Pandharpur.	Analysis and Design of Multistorey Building for Defferent Grades of Steel in Various Seismic Zones	**Approved with Modified title** **"Comparative study of RCC multi-storey building for various grades of steel in various seismic zones"**
9.	**Mr. More Sajjanrao Shivajirao** SVERI's College of Engineering, Pandharpur, Solapur.	Aug-2019	**Dr. M. M. Pawar** SVERI's College of Engineering, Pandharpur.	Progressive Collapse Analysis of RCC Framed Structure	**Approved**
10.	**Miss. Thite Trupti Somnath** SVERI's College of Engineering, Pandharpur, Solapur.	Aug-2019	**Dr. P. M. Pawar** SVERI's College of Engineering, Pandharpur.	Strengthening of cantilever wall against lateral loading	**Approved**
11.	**Mr. Aiwale Hari Dharma** SVERI's College of Engineering, Pandharpur, Solapur.	Aug-2019	**Dr. V. S. Kshirsagar** SVERI's College of Engineering, Pandharpur.	To Determine the Alternative Materials for River Sand To Apply in Mortar for Brick Masonry	**Approved with Modified title** "Identification of the Alternative Materials for River Sand for Brick Masonry Construction"
12.	**Mr. Koshti Ganesh Kumar** SVERI'S College of Engineering, Pandharpur, Solapur.	Aug-2015	**Dr.M. M. Pawar** SVERI'S College of Engineering, Pandharpur.	Comparative Study of Structural Responses of Floating Column and Non-Floating Column Structural Frames	**Approved with Modified title** "Comparative Study of Structural Responses of Structural Frames with Floating Columns"
13.	**Mr. Shubham M. Jadhav** SVERI'S College of Engineering, Pandharpur, Solapur.	Aug-2015	**Dr. S. A. Gosavi** SVERI'S College of Engineering, Pandharpur.	Progressive Collapse Analysis of Reinforced Concrete Multistoried Frame Structure	**Approved**

(Deputy Registrar)
Research Department (Ph.D.-I)

SHRI VITHAL EDUCATION & RESEARCH INSTITUBE'S

COLLEGE OF ENGINEERING, PANDHARPUR.

Department of Civil Engineering

Dessertation Phase-II Progress Seminar

Acad Year:2019-20 **Class-ME (2nd Year)**

Sr. No.	Seat No	Name of Student	Disertation Phase II: Progress Seminar
1	825139	BAGAL SRUSHTI CHANDRAKANT	97
2	825140	JADHAV SUREKHA MAHADEV	97
3	825141	INGALE SUJATA JANARDHAN	97
4	825144	ABHANGRAO CHAITALI RAJENDRA	97
5	825146	MANE YOGESH GURURAJ	92
6	825147	LACHYAN POOJA SHIVANAND	97
7	825148	MAHAMUNI NISHIGANDHA VAIJINATH	97
8	825151	SHAHABADE VIKRANT BABURAO	90
9	825152	PATIL SWAPNIL MOHAN	97
10	825153	PAWAR PRASAD SHIVAJI	94
11	825155	GADE SHUBHAM RAJABHAU	90
12	825157	DESHMANE SHWETA ANIL	92
13	825158	AIWALE HARI DHARMA	90

(Dr. P.M. Pawar)
H.O.D. Civil Dept.

HEAD,
Dept. of Civil. Engg.
C.O.E. Pandharpur

Sample Synopsis Report

A

Synopsis On

"Analysis of Cement Performance Using Microscopic Analysis Techniques"

Submitted to

पुण्यश्लोक अहिल्यादेवी होळकर सोलापूर विद्यापीठ

॥ विद्यया संपन्नता ॥

Punyashlok Ahilyadevi Holkar Solapur University

Submitted by

MISS. CHAITALI RAJENDRA ABHANGRAO

For partial fulfillment post-graduation in M-tech Structure Civil Engineering as per Requirement **Punyashlok Ahilyadevi Holkar Solapur University**

For the academic year 2019 –20

Under the Guidance of

Dr.P.M.Pawar

SVERI

SVERI'S College Of Engineering ,Pandharpur

SVERI'sCOLLEGE OF ENGINEERING, PANDHARPUR.

Gopalpur -Ranjani Road, Gopalpur, P.B. No. 54, Tal - Pandharpur- 413 304,
Dist. Solapur (Maharashtra)
(Approved by AICTE, New Delhi and affiliated to PAH Solapur University, Solapur)
Year -2019-20

CERTIFICATE

This is to certify that the synopsis entitled

"Analysis of Cement Performance Using Microscopic Analysis Techniques"

submitted by

MISS. CHAITALI RAJENDRA ABHANGRAO

is hereby scrutinized and approved by Civil Engineering departmental PG synopsis approval committee and forwarded toPunyashlokAhilyadeviHolkarSolapur University authorities for approval of PG synopsis.

Dr. P. M. Pawar	**Prof. M. M. Pawar**	**Prof. S. A. Gosavi**
(Guide)	(Committee Member-I)	(Committee Member-II)

(Panel of PG Synopsis approval Committee)

B.Ronge

Dr. V. S. Kshirsagar	**Dr. P. M. Pawar**	**Dr. B. P. Ronge**
(PG Coordinator, civil)	(HOD, civil)	(Principal)

Synopsis Presentation And Review Report

(By the college department committee)

M.Tech- Structure (Civil Engineering)

Part-II Semester-III

Name of College :SVERI's College of Engineering, Pandharpur.

Name of Student : Miss. Chaitali Rajendra Abhangrao.

Admission Year (To First Year): 2018

Title of Dissertation: "Analysis of Cement Performance Using Microscopic Analysis Techniques"

Synopsis Presentation Date:

The student has presented the synopsis and is recommended for submission toPunyashlokAhilyadeviHolkar Solapur University, Solapur.

(a) With necessary correction for the following suggestions.

(b) Suggestions given by committee:

Dr. P. M. Pawar	**Prof. M. M. Pawar**	**Prof.S. A. Gosavi**
(Guide)	(Committee Member-I)	(Committee Member-II)

Suggestions Implemented and Reviewed:

Synopsis of above student has been reviewed & recommended for approval after necessary corrections as per the above suggestions for submission toPunyashlokAhilyadeviHolkar Solapur University, Solapur.

SYNOPSIS:

1. Name of the College : SVERI's College of Engineering, Pandharpur.

2. Name of the Course : M.Tech.(Structure) Civil Engineering

3. Name of the Student : Miss. Chaitali Rajendra Abhangrao.

4. Month of Registration : August 2018

5. Name of the Guide : Prof.Dr. P. M. Pawar.
HOD of Civil Department

6. Proposed Title of the Dissertation : **"Analysis of Cement performance Using Microscopic Analysis Techniques."**

INTRODUCTION:

Cement is the binding material. Portland cement is manufactured by crushing, milling and proportioning the following materials Lime or calcium oxide, CaO: from limestone, chalk, shells, shale or calcareous rock. Silica, SiO_2: from sand, old bottles, clay or argillaceous rock. Alumina, Al_2O_3: from bauxite, recycled aluminum, clay. The Portland cement represents a mixture of clinker and finely ground gypsum, where the clinker is made up of four main mineral components Alite (C3S),Belite(C2S),Celite I (C3A), Celite II (C4AF). Portland cement concerning the cement hydration processes performed at various time intervals using XRD. Portland cement Hydration Process highlighted the presence of hydration compounds as well as of mineral compounds with the help of two methods XRD and SEM.The methods used X-ray diffraction and Scanning electronic microscopy applied to define materials and to understand the changes occurring in mineral compounds (alite, belite, celite and brownmillerite) during their modification into hydrated mineral compound. X-ray diffraction method (XED) is used in hydration compounds for identification of the mineral phases, Scanning electronic microscopy (SEM) is powerful technique used in examination of material properties mostly used in metallurgy , geology, biology and medicine. The use of the mentioned methods allows more accurate information regarding the behaviour of the Portland cement paste during hydration, and a more realistic knowledge of the mechanisms that generate new properties such as strength and durability, which are among the most important in the selection of cement for a specific application.

PRESENT THEORIES AND PRACTICES:

Some of the recent and most relevant works are summarized below:

Serban,L.[1] mentioned that XRD and Scanning Electron Microscopy (SEM) gives accurate information regarding to the Portland cement past during the hydration and provided knowledge of few new properties such as strength and durability.At a maximum temperature of up to 1450°C. In the clinker, the calcium silicates represent 75 - 80 %, hence the name of silicate cements, while calcium aluminates and calcium aluminoferrite form only 20 – 25 % .

Neville [2] has presented a review the cement hydration products are poorly soluble in water. The hydration processes of cement particles was not completed immediately speed of hydration gradually diminishes. According to his study the Alite (C3S) is found in cement in largest ratio (50%). Belite (C2S) is the mineral component that has the form of crystal in clinker when slowly cooled down.

Ionescu et al.[3] mixtured of clicker and finely ground gypsum in the Portland cement, where clinker was made of four minerals component Alite (C3S), Belite(C2S),Celite I (C3A),Celite II (C4AF) . The methods used concern X-ray diffraction and electronic microscopy applied to define materials and to understand the changes occurring in mineral compounds (alite, belite, celite and brownmillerite) during their modification into hydrated mineral compounds (tobermorite, portlandite and etringite).

Molnar et al.[4] The hydration and hydrolysis reactions of the two mineral compounds above also produce hydrosilicates that initially have a gel structure similar to that of the natural mineral called tobermorite. The calcium silicate hydrates form the majority of the hydration products, present a gel structure, where the solid phase is made up of a lattice of microcrystals, initially of angstrom size with eyes filled with a saturated composition of components: in a later stage, the crystals develop, age and strengthen, leading to the increase of the mechanical strength

Jumate et al.[5] studied about Hydration Process which highlighted the crystallographic texture, size of the crystallite, internal stresses in sample. Phase qualitative and quantitative analysis also be done by using XRD. If the respective phase shows 3-4% or more than that ,then crystalline phase identify byBragg's relationship and other also.

OBJECTIVE:

- To design experiments for studying the influence of curing parameter attainment of cement strength performance.
- To design experiments for studying the influence of various chemical admixtures on cement performance.
- To perform microscopic analysis of cement mortar with and without cured and compares it with strength results.
- To perform microscopic analysis of cement mortar to understand influenced chemical admixtures and related to strength.

METHODOLOGY:

- ***To design experiments for studying the influence of curing parameter attainment of cement strength performance.***

  Preparation of cement mortar specimen size (7x7) cm. We are going to prepare 32 number of specimen. From total specimens half of specimens we will kept for hydration in curing tank and remaining half (16 specimen) kept for non curing test. For achieving the result we choose the time period as 1, 3, 7, 14 and 28 days. Select randomly any three specimens for UTM test at each decided time period. Note down best result from specimen. (Mentioned procedure is carried out for both the part curing and non curing.)

- ***To perform microscopic analysis of cement mortar with and without cured and compares it with strength results.***

  From strength result preserve some part for SEManalysis. Select any one specimen for each day testing.Take few sample powder from above selected specimen for XRD analysis. Mentioned procedure carried out upto the 28 day hydration. (Mentioned procedure is carried out for both the part curing and non curing.) And compare the result.

- ***To design experiments for studying the influence of various chemical admixtures on cement performance.***

  Procedure is same as mentioned in above two point.Add some Plasticizers while preparing cement mortar specimen (7x7)cm. The time period is fixed as 1,3,7,14 and 28 days. Check out the strength and mineral component for both the cases curing and non curing process .

- ***To perform microscopic analysis of cement mortar to understand influenced chemical admixtures and related to strength.***

  Same way SEM analysis will be carried from specimen in which chemical admixtures is used. Compare the result with previous test and reult after adding admixture.

MATERIALS:

- Portland cement
- Distilled water
- XRD/SEM machine
- UTM machine
- Various types of Admixtures

XRD Machine

Outline of the Proposed Work:

Phase	Type of Work	Expected Period
1.	Getting introduced with new Microscopic Techniques and a detailed literature survey and study of other similar projects.	1 Months
2.	To find out XRD of cement powder at different time interval and work on Analysis.	1.5 Months
3.	Casting of cement mortar (specimen) and find out the strength on UTM Machine and simultaneously working on Microscopic Techniques after 1 ,3 ,7,14 and 28 days on curing and non curing specimens.	1.5 months
4.	Comparative study of UTM result and Microscopic result Working on analysis.	1 month
5.	With the addition of various admixtures prepare the specimens to understand influence, test it after 1,3, 7, 14 and 28 days on curing and non curing specimens. And find out the strength and SEM/XRD result.	1.5 Months
6.	Comparative study of Microscopic result Working on analysis.	1 Months
7.	Linking of all the subtopics of the work appropriately and writing the final report.	1.5 months
Total Duration		**9 Months**

AVAILABLE FACILITY:

The following facilities are available at SVERI's College of Enginnering Pandharpur to carry out dissertation work

1) Laboratory facility and Computing facility
2) Books and IEEE journals, Science direct

EXPECTED DATE OF COMPLETION OF WORK: May 2020

REFERENCES:

1.SERBAN, L. (1998), Materiale de construcii (Building materials), Matrix, Bucureti.

2.NEVILLE, A.M. (2003), Proprietile betonului (Concrete properties), Editura Tehnic, Bucureti.

3.IONESCU, I. and ISPAS, T. (2008), Proprietile i tehnologia betoanelor (Concrete properties and technology), Editura Tehnic, Bucureti.

4.MOLNAR, L., MANEA, D. and ACIU, C. (2010), The study of hydration processes of cement based on latest generation methods, Proceedings of the Internaional Scientific Conference, CIBv 2010, Vol. 1, (November 2010), pp. 215-221, ISSN 1843-6617, Transilvania University Press.

5.JUMATE, E. and MANEA, D.L. (2011), X-Ray difraction study of hydration processes in the Portland cement, Journal Of Applied Engineering Sciences, Vol. 1(14), Issue 1, pp.79-86.

(Ms.Chaitali Rajendra Abhangrao)

Student

(Prof. Dr. P. M. Pawar)

Guide

Date: 28/08/2019.

Place : Pandharpur

<u>Sample Progress Report</u>

Progress Report

“Design and Analysis of Slabs with Insertion Voids For Weight Minimization.”

Ms. Mahamuni Nishigandha Vaijinath

Institute: SVERI’s College of Engineering, Pandharpur

Department: Civil Engineeering

Course: M. Tech. (Structures)

Academic Year: 2019-20

Guide: Dr. P. M. Pawar

I. INTRODUCTION

General

A slab is a structural element, made of concrete, which is applied to create flat horizontal surfaces such as floors, roof decks, and ceilings. A slab is mainly various inches thick and strengthened by beams, columns, walls, or the ground. In building construction, the slab plays a main structural member. The slab is one of the main parts which consume large concrete. The high weights of concrete cause problems for the concrete slab and also decrease the span length. This is why important improvements in R.C.C have concentrated on improving the span, decreasing the weight, or reducing the natural stress defects of concrete. For this cause important branches of reinforced concrete have focused on becoming the span reducing the weight or overcoming concrete's natural tendency in tension. Voids are used to eliminate the undesirable concrete and reduce the weight of the slab, which improves the structural capacity of the slab and also increase the span length. For the self-weight and raise the stability of the slab we use voids in the slab. However, improving people’s enthusiasm in the residential environment, improves noise, vibration, and the deflection of the slab increase by increase span length, which resulted in raising the slab thickness. This increases the use of construction material such as concrete and steel in the construction. To avoid these disadvantages which were caused by increases in the self-weight of slabs, the voided slab system is used.

Systems of Plastic Voided Slab: Plastic voided slab systems are an option for solid concrete slab construction. Plastic voided slab systems are significantly less than solid concrete slabs while keeping the strength to have large spans. Slabs are extra comfortable because less concrete is used in voided slab construction than solid slab construction. Plastic voided slab methods were first introduced in Europe in the 1990s. After that time, many European companies have patented their systems. Various Systems are as follws:

1. Bubble Deck: In the middle of the 1990s, a new system was produced in Denmark by JorganBreuning to secure the loss of dead weight with more than 30% and giving longer spans between supports which is called bubble deck.

Fig.1 Bubble Deck

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2. Cobiax : The same hollow slab system of creating voids within the concrete slab to lighter the building structure was developed in 1997 South Africa, which was called the cobiax system.

Fig.2 Cobiax System

3. U-Boot Beton: U-Boot Beton a new system of hollow formers to decrease the transportation cost and CO2 product was in 2001 by an Italian engineer, Roberto llGrande. U-Boot Beton, or U-Boot, is a voided slab system from the Italian company Daliform.

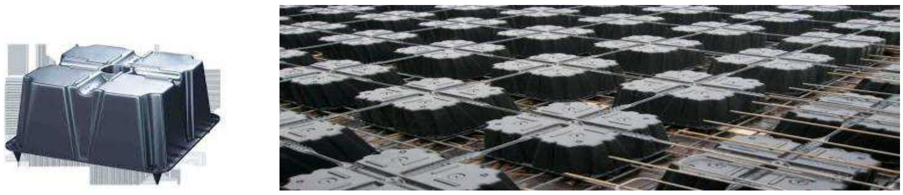

Fig.3 U-Boot Beton System

4 Airdeck: The design, developed in 2003, is Airdeck. The concept of Airdeck is similar to the U-Boot system. The basic uses of Airdeck's former, the use of recycled polypropylene for providing irretrievable void formers that give to preserving the environment.

Fig.4 Airdeck System

1.4.5 Bee Plate System: The BEEPLATE Honeycomb Floor is enough determination for wide span reinforced concrete flat slabs with any suspension. Spans between up to 20 m with floor depths between 34 cm and 70 cm can be provided. By using buoyancy free hollow bodies, construction is particularly easy.

Fig.5 Bee Plate System

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Voided slabs are lightweight when compared to traditional slab construction. Voided slab systems can decrease a dead load of slabs by as much as 35%. The amount of concrete used in the voided slab is 35-50% less than the traditional solid slab. The voided slab is a more time-saving, cost-saving, saving in transportation, material, and labor.

II. LITERATURE REVIEW

Pandharipande and Pathak [1] explained the voided slab by using HDPE balls. The article explained the range of the studies including assessing the flexural strength and performance of voided slab and conventional slab by analytical and experimental work. The slab specimens cast and with 3 kinds Traditional slab, Bubble deck slab and Analysis of slab specimens was done by ANSYS WORKBENCH 16.0 of FEM analysis. It was shown that the B.D.S in practice more beneficial in saving the concrete because of weight reduction. Tandale et.al [2] this paper reviewed several studies done on the voided slab method. All technical parameters of the voided slab method on which experimental studies carried out by authors were listed in this paper systematically. In this paper, the authors compared self-weight of the conventional slab and voided slab with U-Boot Beton respectively. Ghalimath et.al [3] this article explained the behavior of the voided slab by the U-Boot Beton. This section briefly described U-Boot Beton, parts of U-Boot, size of U-Boot. Saranya & Sankaranarayanan [5] this article presented the ultimate load-carrying capacity of slabs with high volume fly ash replacement and also the incorporation of plastic balls in it. This also decreases the overall cost of construction. Shinde et.al [6]this document discussed the comparative research of Flat Plate Slab and Voided Slab Lightened with U-Boot Beton. In this section the design process for flat plate slabs was compared with voided slabs lightened with U-Boot Beton. Pande et.al [7] the report discussed the different structural behavior of voided slab of bubble deck slab and their structural benefits over a traditional concrete slab. In this paper, a Bubble Deck slab has 2D arrangements of voids within the slab to decrease self-weight. In this paper, the bubble deck slab reduces the dead weight, and this reduction discussed by the numerical problem. And also bubble deck slab and flat slab self-weight calculation are calculated manually with an approximated cost. Gore et.al. [8] this paper reviewed several studies done on the voided slab. The comparative study is performed between the analysis of voided and conventional slab method. Ali Omar [9] this paper reviewed several studies done on the voided slab system. The behavior of voided slabs was simulated numerically and carried out experiments on slab specimens with different void shapes, numbers geometry, and reinforcement state. Voided slabs were recommended for roofing and flooring because of lesser self-weight, ease of construction and lighter supporting systems, foundations are required. Naik & Joshi [10] this paper discussed the deal with work carried out to compare the bubble deck voided slab system and conventional flat slab system by finite element analysis using SAP 2000. An additional study was carried out to investigate the seismic behavior of structure due to a decrease in the dead weight of the structure by modeling and analysis of G+12 story structure for 6mx 7m, 7m x 8m, 8mx 9m grid operations. The findings may show that the slabs momentum is condensed by 7 to 10%of the strong flat slab. Yadav & Tambe [11] this paper discussed the studies on the design method for the plastic voided slab is compared with reinforced concrete solid flat slab through a design comparison of typical bays. Bhagat & Parikh [12]the paper presented the various parameters of the voided and solid flat plate slabs are calculated to relate both systems. To evaluate the appearance of the R.C.C voided and solid flat plate slabs, modeling of slabs is carried out using SAP 2000 with different spans.

III. MATERIAL AND ITS PROPERTIES

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1 **Selection of Materials:** The voided slab is composed mainly of steel, plastic, P.C.C., and concrete:

Table 1

Material Properties

Sr. N0.	Name of Material	Property	Value
1.	**P.C.C. (M15 grade)**	Modulus of Elasticity (E) in MPa.	22630
		Density of Concrete(ρ) in Kg/m^3	2400
		Poisson's Ratio(μ)	0.167
		P.C.C. Compressive Ultimate Strength in MPa.	15
2.	**Concrete I)For M20 Grade**	Modulus of Elasticity (E) in MPa.	22360.679
		Density of Concrete(ρ) in Kg/m^3	25000
		Poisson's Ratio(μ)	0.2
		Compressive Ultimate Strength in MPa	20
	II)For M25 Grade	Modulus of Elasticity (E) in MPa.	25000
		Density of Concrete(ρ) in Kg/m^3	25000
		Poisson's Ratio(μ)	0.2
		Compressive Ultimate Strength in MPa	25
	II)For M30 Grade	Modulus of Elasticity (E) in MPa.	27386.127
		Density of Concrete(ρ) in Kg/m^3	25000
		Poisson's Ratio(μ)	0.2
		Compressive Ultimate Strength in MPa	25
4.	**Steel**		
	I)Fe415 Grade	Modulus of Elasticity (E) in MPa.	20000
		Density of Concrete(ρ) in Kg/m^3	7850
		Poisson's Ratio(μ)	0.3
		Tensile Yield Strength in MPa	415
	I)Fe500 Grade	Modulus of Elasticity (E) in MPa.	$2.18X10^{11}$
		Density of Concrete(ρ) in Kg/m^3	7850
		Poisson's Ratio(μ)	0.3
		Tensile Yield Strength in MPa	500
5.	**HDPE Balls**	Modulus of Elasticity (E) in MPa.	950
		Density of Concrete(ρ) in Kg/m^3	1030
		Poisson's Ratio(μ)	0.4

IV. ANALYTICAL STUDY

The effective dimension of the P.C.C. slab is 3150mmX4200mmX210mm and for voided slab No. of voids=9 with the same dimension. And the diameter of the void is 120mm. With the slab fixed supported at four edges in both cases. Assume the density of concrete is 2400 Kg/m3. Calculate the weight in concrete and compare.

a) Without Voids Calculation: Dimension of Slab: 3150mmX 4200mm = 3.15m X4.20m Thickness: 210mm=0.21m.

Volume of P.C.C. Slab without Voids (V1) = L X B X t = 3.15X4.20X0.21 = 2.7783m3.
b) With Voids Calculation:

Dimension of Slab: 3150mmX 4200mm = 3.15m X4.20m

Thickness: 210mm=0.21m

No of Voids in P.C.C. Slab = 9No's.

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Diameter of Each Voids=120mm.

Volume of Voids in the Slab (V2) = 9X (Π/4) X d^2 X 4.2

= 9X (Π/4) X 0.12^2 x 4.2

Volume of Voids (V2) in the Slab= 0.4275m^2.

Volume of P.C.C. Slab with Voids V = (V1-V2) = 2.7783-0.4275

Volume of P.C.C. Slab with Voids=2.3508m^3.

Weight Calculation: Weight of P.C.C. Slab without Voids (W1) = (ρc) X V1= 24 X 2.7783 = 66.6792KN

Weight of P.C.C. Slab without Voids (W1) =6667.92 Kg.

Weight of P.C.C. Slab with Voids (W2) = (ρc) X V= 24 X 2.3508 =56.4190KN=5641.90Kg.

Weight Reduction in the Slab (%):

Weight Reduction = 100 - (W2/W1) X 100= 100 - (5641.90/6667.92) X 100

Weight Reduction= 15.387%

For R.C.C. Slab:

Similarly in R.C.C. Slab With and Without Voids the weight Reduction in % = 25.65%

Modelling

A. Engineering Data

To have the model of correct behavior it's important to define the type of material, along with its mechanical properties using the Engineering Data cell. The model deformation response will depend on the material defined. The properties of Slab entered in the engineering data cell are shown in Fig.

B. Geometry

To create the geometry of Slab we are going to use the Design Modeler program. Using the Design Modeler we will 'draw' our system part(s) using basic drawing tools and then extrude our 2D cross-sections into 3D elements. Fig. shows the geometry of the Slab.

C. Material Assignment

A new mechanical window allows the assignment of material. In the this case, there is only one material color shown. The assignment of material is shown in Fig. .

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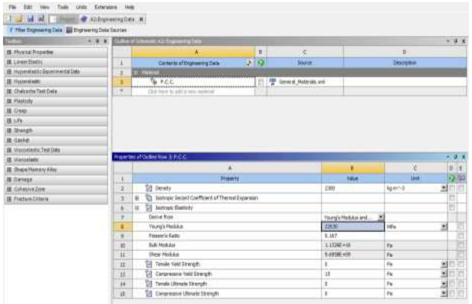

Fig 6 Engineering Data Window with Properties of S355

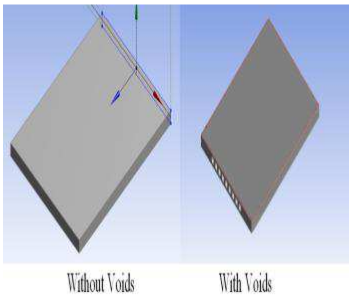

Fig. 7 Geometry

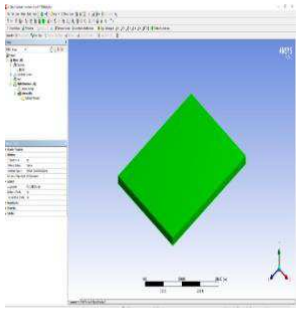

Fig. 8 Material Assignment

D. *Meshing*

After assigning the material, the mechanical window also allows working on the meshing of a slab with and without voids. A mesh is needed to run a Finite Element Analysis (FEA). The mesh takes the 3D part and represents it as many small elements that are connected by nodes. The FEA cannot run without having a mesh defined. And once the mesh is generated it will appear on 3D as shown in Fig. 4.

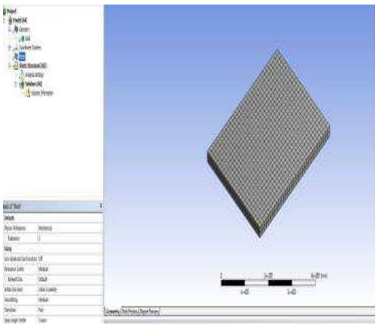

Fig. 9 Meshing

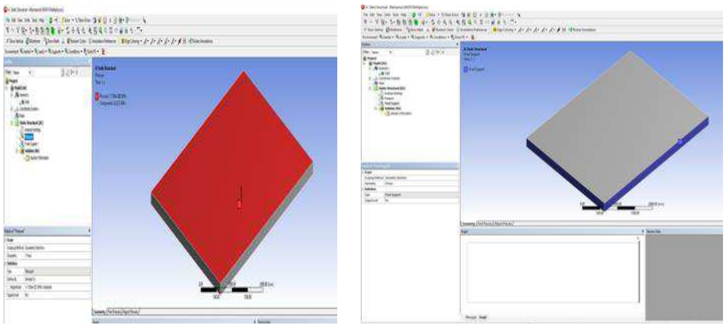

Fig. 10 Loading and Support Condition

E. *Setup*

Use the Setup cell to launch the appropriate application for that system. Using these options we can define loads and boundary conditions. For our current system, we are going to apply a concentrated force of 4000 KN and fixed support at the bottom which is shown in Fig. 5.

F. *Results*

To run the analysis and to set-up a viewer for the Axial Deformation we are going to select Solution->Insert->Deformation- >Total. Similarly to set-up a viewer for the Equivalent Stress we are going to select Solution->Insert->Stress->Equivalent Stress.

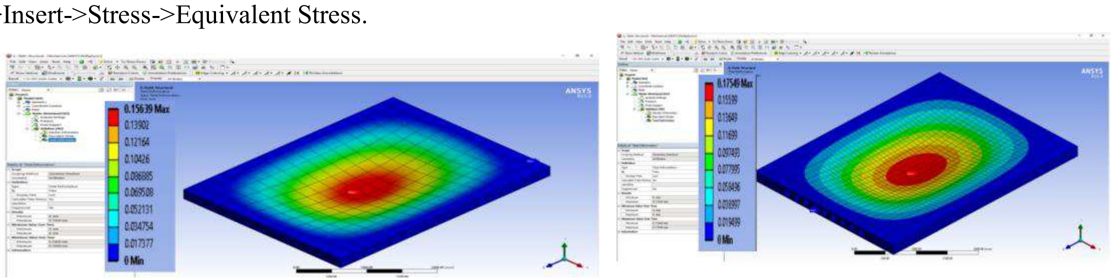

Fig. 11 Total Deformation (mm)

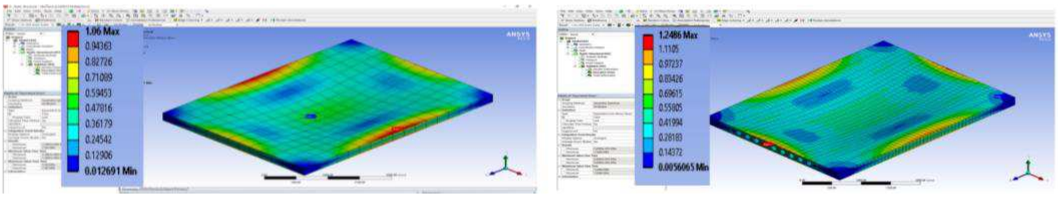

Fig. 12 Equivalent Stress (MPa)

V. RESULTS

After analyzing the slab with the same geometry and support condition subjected to different loads, the results are obtained are shown in Table 2. And also for P.C.C. and R.C.C. slab with and without HDPE Balls as voids.

Table 2

Variation of maximum equivalent stress and total deformation with varying thickness

PCC Slab with Varying Thickness					
Sr. No.	Thickness (mm)	Without Voids		With Voids	
		Maximum Equivalent Stress (MPa)	Maximum Total Deformation (mm)	Maximum Equivalent Stress (MPa)	Maximum Total Deformation (mm)
1	180	1.233	0.2102	1.4299	0.2438
2	190	1.2	0.1944	1.4264	0.2224
3	200	1.206	0.1738	1.3159	0.1964
4	210	1.06	0.1563	1.2486	0.1754
5	220	0.967	0.1366	1.1185	0.1524
6	230	0.917	0.1242	1.0608	0.1381
7	240	0.872	0.1138	1.0217	0.1257
8	250	0.832	0.1045	0.9751	0.1153

Table 3

Variation of maximum equivalent stress and total deformation with different grade

RCC Slab with Varying Grades				
Grade of Concrete and Steel	Without Voids of HDPE Balls		With Voids of HDPE Balls	
	Equivalent Stress Maximum (MPa)	Total Deformation Maximum (mm)	Equivalent Stress Maximum (MPa)	Total Deformation Maximum (mm)
M20 & Fe415	0.088602	0.0004635	0.087502	0.0004735
M25& Fe415	0.0888	0.0004146	0.087602	0.0004146
M30 & Fe500	0.089512	0.0003457	0.088602	0.0003785

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VI. CONCLUSION

8.1 Conclusion

The following are the various conclusions occur in this study as per analysis:

1. The voided slab technology is very advanced, economical, and fastest method of construction of a slab. The usage of this technology is very rare due to a lack of awareness in our country.

2. The weight of concrete in P.C.C. slab by insertion of voids along length reduces the weight up to 15.387%. Due to Concrete usage is reduced, this avoids the cement production. And also reduces the cost of cement.

3. The weight of concrete in R.C.C. slab by insertion of HDPE Balls as voids reduces the weight up to 25.65%. Due Concrete usage is reduced, by Recycled HDPE plastic balls replaces the concrete. This avoids the cement production and allows a reduction in global CO_2 emission. Hence this technology is environmentally green and sustainable. And Step towards Green Building.

4. The Stress and deformation coming on the voided slab in P.C.C. and R.C.C. slab are within the permissible limit. The deformation of the voided slab is found to be more than the solid slab in both P.C.C. and R.C.C. slab.

5. The ideal solution for creating slabs with great load - bearing capacity.

8.2 Future Scope

- Voided slab allow architectural freedom of design with non - rectilinear plan forms. Instead of rectangular and square old shape slabs we got the liberty to design any shape as per our design any shape as per our desire thus enhancing the aesthetic view of the structure.

- The ideal solution for creating slabs with large span.

- It is particularly suited for structures that required considerable open spaces, such executive, commercial and industrial buildings as well as public, civil and residential structure.

A

DISSERTATION

ON

"Structural Behaviour Of Voided Beam."

Submitted to

Punyashlok Ahilyadevi Holkar Solapur University, Solapur

Submitted by

Mr.Shubham Rajabhau Gade

In the partial fulfillment for the award of
M. Tech. (Structure)

Under the Faculty of Engineering and Technology

Under the Guidance of

Prof. M.M. Pawar

Department of Civil Engineering

SVERI

SVERI's COLLEGE OF ENGINEERING, PANDHARPUR

2019 - 2020

SVERI's COLLEGE OF ENGINEERING, PANDHARPUR

CERTIFICATE

This is to certify that the synopsis entitled

"Structural Behaviour Of Voided Beam."

submitted by

Mr.Shubham Rajabhau Gade

Is here by scrutinized and approved by Civil Engineering Departmental PG synopsis approval committee and forwarded to Punyashlok Ahilyadevi Holkar Solapur University, Solapur authorities for approval of PG synopsis.

(Prof. M. M. Pawar)	**(Prof. S. A. Gosavi)**	**(Dr. V. S. Kshirsagar)**
(Guide)	**(Examiner-I)**	**(Examiner-II)**

(Panel of PG Synopsis Approval Committee)

(Dr. V. S. Kshirsagar)	**(Dr. P. M. Pawar)**	**(Dr. B. P. Ronge)**
P. G. Co-ordinator	**Head, Civil Engg. Dept.**	**Principal**

A

DISSERTATION

ON

" Comparative study of strength properties of Green Concrete and Conventional Concrete"

Submitted to

Punyashlok Ahilyadevi Holkar Solapur University, Solapur

Submitted by

Mr.Prasad Shivaji Pawar

In the partial fulfillment for the award of

M. Tech. (Structure)

Under the Faculty of Engineering and Technology

Under the Guidance of

Prof. Mukund M. Pawar

Department of Civil Engineering

SVERI

SVERI's COLLEGE OF ENGINEERING, PANDHARPUR

2019 - 2020

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CERTIFICATE

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*is hereby scrutinized and approved by **Civil Engineering Departmental** PG synopsis approval committee and forwarded to Punyashlok Ahilyadevi Holkar Solapur University authorities for approval of PG synopsis.*

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DISSERTATION

ON

"Ultrasonic Pulse Velocity Performance Analysis of Concrete Under Various Conditions."

Submitted to

Punyashlok Ahilyadevi Holkar Solapur University, Solapur

Submitted by

Ms. Sujata Janardhan Ingale

In the partial fulfillment for the award of
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Sample Thesis

A

DISSERTATION REPORT

ON

"PERFORMANCE BASED SEISMIC DESIGN ANALYSIS OF STEEL FRAMES"

SUBMITTED IN PARTIAL FULFILLMENT

FOR THE AWARD OF

MASTERS OF TECHNOLOGY

IN

STRUCTURAL ENGINEERING

PUNYASHLOK AHILYABAI HOLKAR SOLAPUR UNIVERSITY, SOLAPUR

SUBMITTED BY

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"PERFORMANCE BASED SEISMIC DESIGN ANALYSIS OF STEEL FRAMES"

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MS. SRUSHTI CHANDRAKANT BAGAL

Is hereby scrutinized and approved by Civil Engineering Departmental PG synopsis approval committee and forwarded to Punyashlok Ahilyadevi Holkar Solapur University, Solapur authorities for approval of PG synopsis.

(Dr. P. M. Pawar)	**(Prof. M. M. Pawar)**	**(Dr. V. S. Kshirsagar)**
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P. G. Co-ordinator	**Head, Civil Engg. Dept.**	**Principal**

DECLARATION

I hereby declare that dissertation entitled **"PERFORMANCE BASED SIESMIC DESIGN ANALYSIS OF STEEL FRAMES"** completed and written by me has not previously formed the basis for the award of any degree or diploma or other similar title of this or any other university or examining body.

Place: Pandharpur.

Date:

Bagal.C.

(Miss. Bagal Srushti Chandrakant)

ACKNOWLEDGEMENT

The satisfaction that accompanies the successful completion of my dissertation work would be incomplete without the mention of people who are responsible for the completion of the project. I wish to express my sincere and heartfelt gratitude to those, whose timely guidance has made me accomplish my dissertation work satisfyingly. It gives me an immense pleasure and satisfaction to express my deep sense of gratitude to my guide **Dr. P. M. Pawar** , Dean Academics and Head, Department of Civil Engineering SVERI's, COE Pandharpur, for his encouragement and guidance towards making this dissertation work successful.

I express my sincere gratitude and thanks to **Prof. M. M. Pawar**, Campus Director, Associate Professor, Department of Civil Engineering, SVERI's COE Pandharpur, for his patience, motivation, valuable advice and support which helped me complete the dissertation work successfully without any hindrance.

I am greateful to **Dr. V. S. Kshirsagar and Prof. Dr. S. A. Gosavi**, Assistant Professor and Assistant Head, Department of Civil Engineering, SVERI's COE Pandharpur, for thier moral support throughout my dissertation work.

I would like to thank the members and all professors, Department of Civil Engineering, SVERI's COE Pandharpur, for their support and guidance throughout the completion of my dissertation work.

It is a pleasure to express my gratitude wholeheartedly to the non-teaching staff of Department of Civil Engineering, SVERI's COE Pandharpur, for their indispensable help during the laboratory work.

It is a pleasure to express my gratitude wholeheartedly to **Er. Laxman N. Kawathe and Er. Sambhaji N. Kawathe** for their valuable advice and help throughout my dissertation work. Finally, I take this opportunity to extend my gratitude and respect to family and friends for their continuous support and guidance and being source of inspiration.

SRUSHTI C. BAGAL

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CHAPTER 1
INTRODUCTION

1.1MOTIVATION

The destructive impact of natural disasters has become increasingly evident over the past decades. The socio-economic consequences of these incidents are important and can take decades to resolve. Fig. 1 displays the losses from early last century due to natural disasters.Since the 80's a major increase of the amount of losses has been observed as one can conclude from the data shown. While this phenomenon can be explained by various factors, it is accepted that certain circumstances play a decisive role:

(i) Growing population density in already densely populated cities in high-risk areas.
(ii) A rise in living standards, with a resulting rise in property and infrastructure prices.
(iii) A lack of sufficient preparedness and readiness to cope with the disaster consequences.

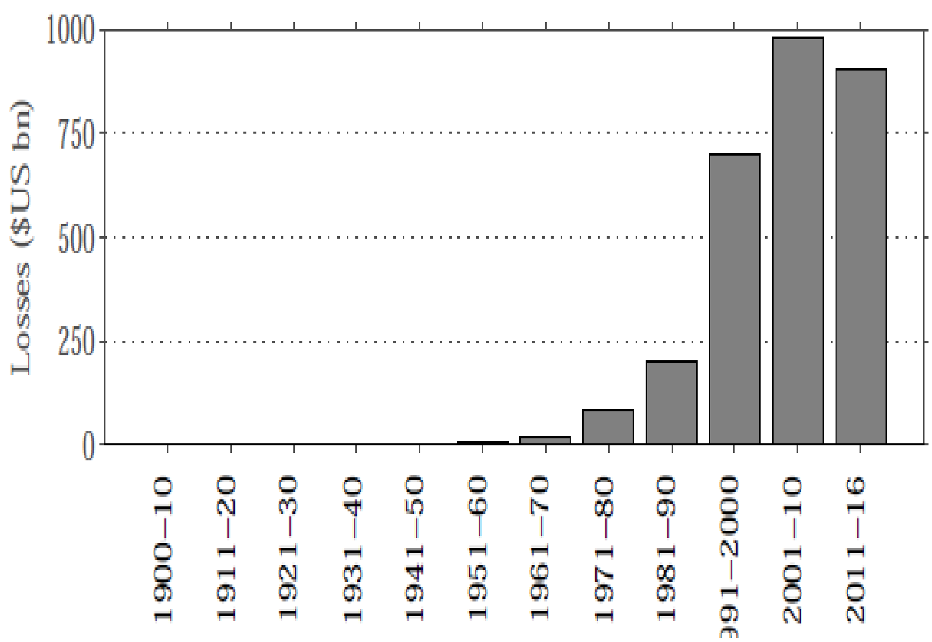


Fig 1. 1Losses due to natural disasters from 1900-2016 (source: http://www.emdat.be/)

Earthquakes are perhaps among the most damaging typologies of natural disasters, with major economic, environmental and cultural implications. Fig. 1.1 displays the categories of the world's most expensive and deadly incidents in recent decades.

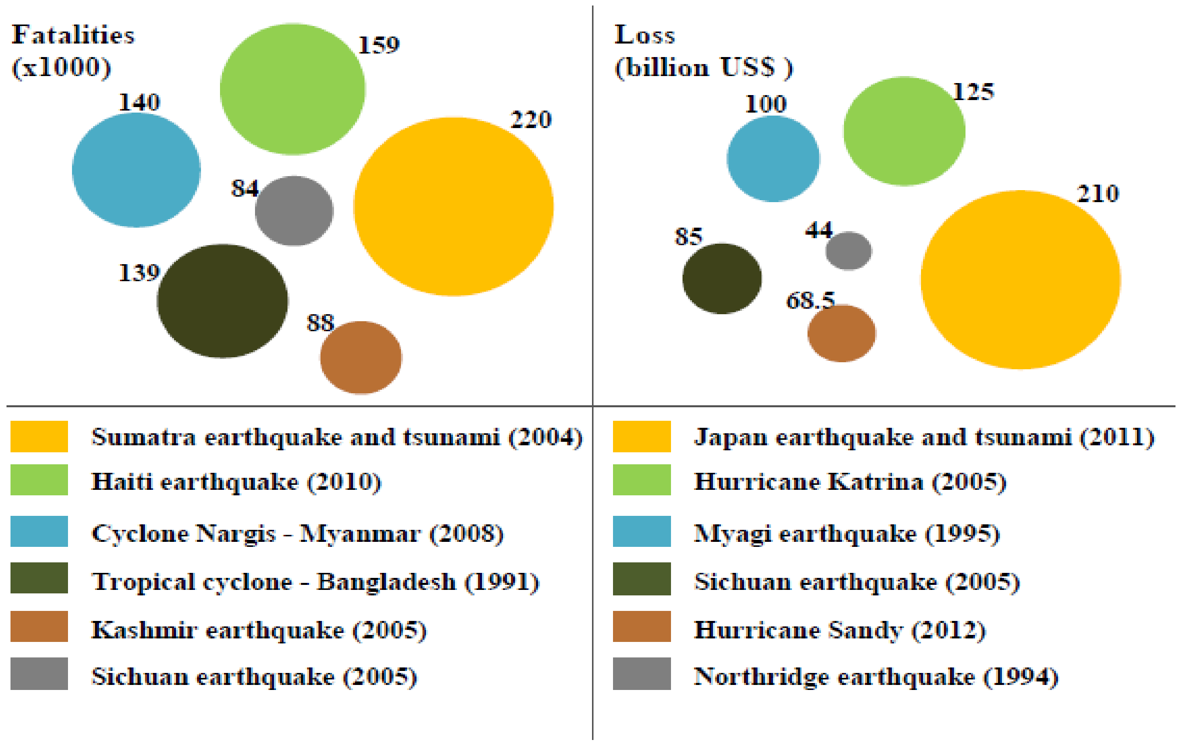


Fig 1. 2Losses due to natural disasters from 1900-2016 (source: http://www.emdat.be/)

As can be observed from Fig.1.2, earthquakes have a remarkable significance in terms of the number of deaths and immediate economic damages among the various forms of natural disasters. In addition, there seems to be no clear connection between the incidents with the highest tolls to human life and the economy, as no natural catastrophe is replicated in both categories shown in Fig.2.These two parameters are in fact closely related to the socio-economic characteristics and growth level of the affected areas. For example, the 2010 Haiti earthquake was responsible for a large number of deaths that could be caused by the lack of appropriate earthquake-resistant construction practices, as well as the insufficient building stock quality. These wide and highly populated urban areas in developing countries placed rapid building and infrastructure growth that typically showed inadequate quality and safety standards. In addition, the lack of preparedness in developed countries for the problems associated with post-earthquake scenarios is also a critical concern regarding the number of fatalities reported.With respect to the earthquakes which caused significant economic losses (e.g. 2011 T¯ohoku, Japan, 1995 Kobe, Japan, 1994 Northridge, USA, 2010 Chile, 2011 Christchurch, New Zealand), the losses were largely associated with the disruption of the sector.Although most of the structures behaved as planned, namely with regard to safeguarding life-safety (as per design requirements), significant damage rates (mainly in the non-structural elements) resulted in significant repair times and, in some cases, inconvenient repair costs.Consequently, the normative design criteria that existed at the time, which concentrated primarily on preventing structural failure, were questioned. For example, during the 2010 Chile earthquake, a substantial number of hospitals (15), within a distance of 500 km to the epicenter,were required to stop operating due to significant non-structural damage.Following table 1.1 provide information about earthquakes occurred in India which suggests the level of disaster caused due to earthquakes.

Table 1. 1History of Earthquakes in India

Date	Location	Mag.	I	Deaths	Injuries	Total damage / notes
2017-01-03	India, Bangladesh	5.7 Mw	V	3	8	
2016-01-04	India, Myanmar, Bangladesh	6.7 Mw	VII	11	200	
2015-10-26	Afghanistan, India, Pakistan	7.7 Mw	VII	399	2,536	
2015-05-12	Nepal, India	7.3 Mw	VIII	218	3,500+	
2015-04-25	Nepal, India	7.8 Mw	IX	8,964	21,952	$10 billion
2013-05-01	Kashmir	5.7 Mw		3	90	$19.5 million
2011-09-18	Gangtok, Sikkim	6.9 Mw	VII	>111		
2009-08-10	Andaman Islands	7.5 Mw	VIII			Tsunami warning issued
2008-02-06	West Bengal	4.3 Mb		1	50	Buildings damaged
2007-11-06	Gujarat	5.1 Mw	V	1	5	Buildings damaged
2006-11-29	Alwar district, Rajasthan	4.0 Mw		1	2	Minor damage to property

Date	Location	Mag.	I	Deaths	Injuries	Total damage / notes
2006-03-07	Gujarat	5.5 Mw	VI		7	Buildings damaged
2006-02-14	Sikkim	5.3 Mw	V	2	2	Landslide
2005-12-14	Uttarakhand	5.1 Mw	VI	1	3	Building destroyed
2005-10-08	Kashmir	7.6 Mw	VIII	86,000–87,351	69,000–75,266	2.8 million displaced
2002-09-13	Andaman Islands	6.5 Mw		2		Destructive tsunami
2001-01-26	Gujarat	7.7 Mw	X	13,805–20,023	~166,800	
1999-03-29	Chamoli district-Uttarakhand	6.8 Mw	VIII	~103		
1997-11-21	Bangladesh, India	6.1 Mw		23	200	
1997-05-22	Jabalpur, Madhya Pradesh	5.8 Mw	VIII	38–56	1,000–1,500	$37–143 million
1993-09-30	Latur, Maharashtra	6.2 Mw	VIII	9,748	30,000	
1991-10-20	Uttarkashi, Uttarakhand	6.8 Mw	IX	768–2,000	1,383–1,800	
1988-08-21	Udayapur, Nepal	6.9 Mw	VIII	709–1,450		

Date	Location	Mag.	I	Deaths	Injuries	Total damage / notes
1988-08-06	Myannmar, India	7.3 Mw	VII	3	12	
1988-02-06	Bangladesh, India	5.9 Mw		2	100	
1986-04-26	India, Pakistan	5.3 Ms		6	30	Severe damage
1984-12-30	Cachar district	5.6 Mb		20	100	Severe damage
1982-01-20	Little Nicobar	6.3 Ms			Some	Moderate damage
1980-08-23	Kashmir	4.8 Ms		Few		Limited damage / doublet
1980-08-23	Kashmir	4.9 Ms		15	40	Moderate damage / doublet
1980-07-29	Nepal, Pithoragarh district	6.5 Ms		200	Many	$245 million
1975-01-19	Himachal Pradesh	6.8 Ms	IX	47		
1970-03-23	Bharuch district	5.4 Mb		26	200	Moderate damage
1967-12-11	Maharashtra	6.6 Mw	VIII	177–180	2,272	$400,000
1966-08-15	North India	5.6		15		Limited damage

Date	Location	Mag.	I	Deaths	Injuries	Total damage / notes
1966-06-27	Nepal, India	5.3 Ms	VIII	80	100	$1 million
1963-09-02	Kashmir	5.3		80		Moderate damage
1960-08-27	North India					Moderate damage
1956-07-21	Gujarat	6.1 Ms	IX	115	254	
1954-03-21	India, Myanmar	7.4 Ms				Moderate damage
1950-08-15	Assam, Tibet	8.6 Mw	XI	1,500–3,300		
1947-07-29	India, China	7.3 Mw				
1941-06-26	Andaman Islands	7.7–8.1 Mw		8,000		Destructive tsunami
1935-05-31	Quetta, Baluchistan	7.7 Mw	X	30,000–60,000		
1934-01-15	Nepal	8.0 Mw	XI	6,000–10,700		
1932-08-14	Assam, Myanmar	7.0 Ms				Moderate damage
1905-04-04	Kangra	7.8 Ms	IX	>20,000		

Date	Location	Mag.	I	Deaths	Injuries	Total damage / notes
1897-06-12	Shillong, India	8.0 Mw	X	1,542		
1885-06-06	Kashmir					Severe damage
1885-05-30	Srinagar			3,000		Extreme damage
1881-12-31	Andaman Islands	7.9 Mw	VII			Significant in seismology
1869-01-10	Assam, Cachar	7.4 Mw	VII	2		Severe damage
1845-06-19	Rann of Kutch	6.3 Ms	VIII	Few		Limited damage / tsunami
1843-04-01	Deccan Plateau					Moderate damage
1833-08-26	Bihar, Kathmandu	8.0 Ms				Severe damage
1828-06-06	Kashmir			1,000		Severe damage
1819-06-16	Gujarat	7.7–8.2 Mw	XI	>1,543		Formed the Allah Bund
1618-05-26	Bombay		IX	2,000		Severe damage
1505-06-06	Saldang, Karnali zone	8.2–8.8		6,000		

The steel systems have been a reliable alternative to conventional reinforced concrete approaches over the last few decades. The increasing understanding of environmental protection, combined with improved strength and ductility characteristics, as well as faster construction methods, highlighted the advantages of this structural solution, with attractive and adequate features from a seismic resistant perspective. It is necessary to remember; however, that the use of ductile materials does not automatically result in ductile structures and that robust design criteria enabling the use of all the potential of the material must be met.The degree of damage found in steel structures following the earthquakes in Northridge (USA, America, 1994, 6.7 Mw) and Kobe (Japan, Asia, 1995, 6.9 Mw) prompted the development of research studies and experimental campaigns aimed at improving standards and guidelines for the seismic design of steel structures. Seismic design of steel structures has become a growing research trend in world, particularly since the early 1990s.

From the discussion in the previous section, the value of developing improved design rules becomes apparent, which allow for a reliable prediction of structural performance under seismic loads.

1.2 STEEL

Steel is the most valuable construction material in the world, by far. The steel industry today is the fundamental or main industry in any region. This is about ten times the strength of concrete; steel is the perfect material of modern building. It's mainly benefits are power, erection speed, prefabrication and demount ability. In structures, structural steel is used in load-bearing frames, and as members of trusses, bridges, and space frames. Steel however needs protection against fire and corrosion. Cladding and dividing walls in steel buildings are constructed of masonry or other materials, and a concrete foundation is also supported.Steel is also used in the design of conjoint frames and shear walls. Steel structures tend to be more economical than concrete structures for tall buildings and large span buildings and bridges, due to their high strength to weight ratio. Steel structures can be built very quickly and this makes early use of the structure thereby contributing to the overall production of steel providing much greater compressive and tensile strength than concrete and allowing for lighter construction.

Steel structures should be engineered and secured to withstand corrosion and fire in order to get the most benefit out of steel. They should be planned and comprehensive to be simple to construct and to build. Good quality control is important to ensure that the different structural components are correctly installed. In design the temperature effects should be considered. Steel structures are ductile and strong, and can withstand extreme loads including earthquakes. Steel structures can be patched and retrofitted quickly for handling higher loads.

Steel is one of the most sustainable construction materials for the environment-steel is 100 percent recyclable.The connections and especially the welds should be properly engineered and detailed to prevent cracks forming under fatigue and earthquake loads. Unique steels and corrosion and fire safety measures are available, and the designer should be familiar with the available choices. Since steel is manufactured under better quality control at the factory, steel structures have greater durability and protection.

1.3 HISTORICAL DEVELOPMENT OF STEEL

Steel was used from 3000 BC and used in china and then in Europe between 500-400 BC. In India the steel-made Ashoakan pillar and the iron joints used in Puri temples are over 1500 years old. The modern technology of blast furnace which was built in AD1350 (Guptha 1998)

In the later part of the eighteenth century, the large-scale use of iron for structural purposes began in Europe. The first significant use of cast iron was Darby's 30.4-m-span Coalbroakadale Arch Bridge in England, constructed in 1779 over the Severn River. The use of cast iron persisted until about 1840. Abraham Darby discovered a way in 1740 to turn coal into coke, revolutionizing the method of iron production. HenoryCort discovered a way of wrought iron in 1784, which is heavier, more flexible and has a greater tensile strength than cast iron.Wrought iron chains were used during 1829 in Menai Straits suspension bridge designed by Thomas Telford and Britannia Bridge by Robert Stephenson was the first wrought iron box girder bridge. Steel was first used in 1740, but was not available in large quantities until the method of producing steel was discovered and patented by Sir Henry Bessemer of England in 1855. In 1865, the open-hearth method was discovered by Siemens and Martin and widely used for the manufacture of structural steel. Companies like Dorman Long began rolling steel I-section by 1880.Riveting was used as a means of fastening until about 1950 when it was replaced by welding.Bessemer’s steel production in Britain ended in 1974 and last open –hearth furnace closed in 1980.The basic oxygen steel making (BOS) process using the CD converter was invented in Austria in 1953.Today we have several varieties of steel.

1.4 TYPES OF STRUCTURAL STEEL

The structural designer is now able to choose structural steel from the following standardized categories for a specific application.

A. **Carbon steel (IS 2062):** The primary reinforcing elements are carbon and manganese. The defined minimum ultimate tensile strength for these ranges from approximately 380 to 450 MPa and their stated minimum yield strength from approximately 230 to 300MPa(IS 800:2007).

B. **High –strength carbon steel:**This steel is required for structures such as transmission lines and microwave towers. The required ultimate tensile strength of approximately 480-550 MPa and minimum yield strength of approximately 350-400 MPa.
C. **Medium-and-high strength micro alloyed steel(IS 85000):** This steel has low carbon content but, due to the addition of alloys like niobium, vanadium, titanium, or boron, it achieves high strength. The required final tensile strength, ranging from approximately 440-590 MPa, and minimum yield strength of approximately 300-450 MPa.
D. **High-strength quenched steels and temperature steels(IS 2003):**This steel is heat treated to produce a high resistance. The required ultimate tensile strength of approximately 700-950 MPa, and minimum yield strength of approximately 550-700 MPa.
E. **Weathering steels:** This low-alloy atmospheric corrosion-resistant steel has an overall tensile strength of approximately 480 MPa and an performance of approximately 350 MPa.
F. **Stainless steels:** This steel is essential low-carbon steel adding at least 10.5 percent (max 20 percent) of chromium and 0.5 percent of nickel.
G. **Fire resistant steels:** Also known as thermo-mechanically treated steels, they perform better under fire than ordinary steel.

1.5 PERFORMANCE BASED SEISMIC DESIGN

Recent earthquake caused catastrophic damage in overall world. Steel structures are considered mostly earthquake resistant structure but some significant failures have occurred. Recent earthquake events demonstrate the necessity of change in structural design guidelines. To protect and maintain the economic activity and prosperity of a region, the performance of structure caused by earthquake became a major factor. That’s why Civil Engineering profession is updating structural design paradigm of life safety (LS) to the performance bases seismic design (PBSD). Conventional seismic design approaches have the purpose of ensuring life safety (strength and ductility) and regulation of damage (drift limits for serviceability). The design parameters are specified by the stress limits and the strengths of the members determined from the prescribed lateral shear force.

Performance-based design is a more general design philosophy in which the design criteria are expressed in terms of achieving stated performance objectives when the structure is subjected to stated levels of seismic hazard. The performance targets may be a level of stress not to be exceeded, a load, a displacement, a limit state or a target damage state. There have been different interpretations of what is meant by performance-based design. The most appropriate definition is that performance-based design refers to the methodology in which structural design criteria are expressed in terms of achieving a set of performance objectives.

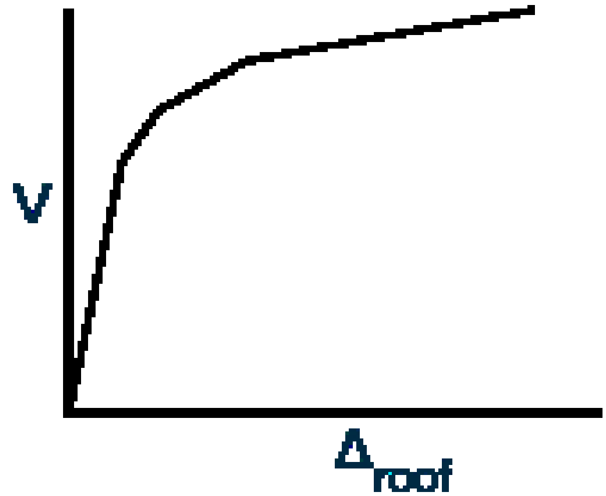
Using an appropriate structural system is critical to good seismic performance of the buildings. While moment frame is the most commonly used lateral load resisting structural system, other structural system are also commonly used such as braced system. A bracing is a system offered to reduce lateral structural deflection. Braced frame virtually eliminates bending factors for the column and girders and thus improve the efficiency of mere rigid frame behavior. Already proved that braced frame decreases the displacement of the structure and absorbs more energy during earthquake. But the study does not comment on the effect of the position of the bracing on the structure. Considering this gap, in this study 3 frames are considered one is moment and remaining 2 are braced frame. In that braced frame one frame is externally braced as concluded that external bracings perform well under lateral loads. Second frame is internally braced with optimum position as was that adding braces to the core of building reduces the drift much more than adding them to the facades. Comparative study of three frames is presented in the study to demonstrate which structural design shows best performance under earthquake loadings.

1.6 NONLINEAR PUSHOVER ANALYSIS

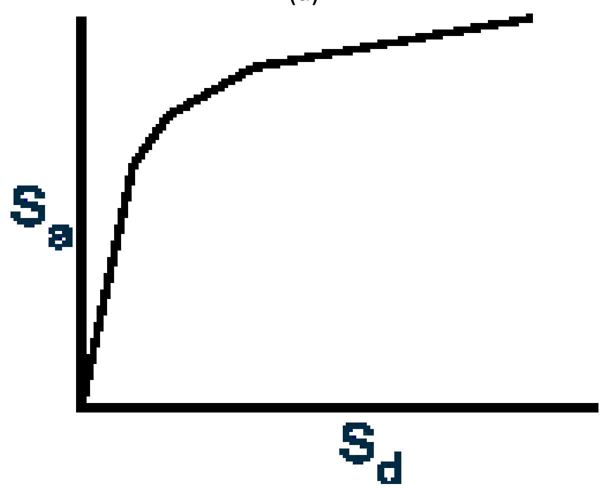
Structural frames considered are analyzed in STAAD Pro advanced by nonlinear static analysis, popularly known as pushover analysis which is one type of PBSD. The nonlinear seismic analysis is used in structural Engineering profession to design steel frames for moderate to strong earthquakes. The linear procedures maintain the traditional use of a linear stress-strain relationship but incorporate material acceptance criteria to permit better consideration for probable non-linear characteristics of seismic response. The non-linear static procedure, often called “pushover analysis,” uses simplified nonlinear techniques to estimate seismic structural deformations. As per FEMA 356, a pushover analysis is a static nonlinear way of estimating seismic structural deformations using a simplified, non-linear technique. Earthquake engineering research is progressing rapidly to consider the nature of buildings that have been exposed to powerful earthquakes. Pushover analysis is done to be able to predict such behavior. The overall capacity of a structure depends on the strength and deformation capacities of the structure's individual components. to evaluate capacities beyond the elastic limit some form of nonlinear analysis is needed, such as Pushover Analysis. It is a modern performance based seismic design (PBSD) for analytically achieving a structural design that will work reliably under one or more seismic conditions in a specified manner. There are two nonlinear procedures using pushover methods: a. Capacity Spectrum Method b. Displacement Coefficient Method. In this analysis particularly Capacity Spectrum Method is used.

1.6.1 Capacity Spectrum Method

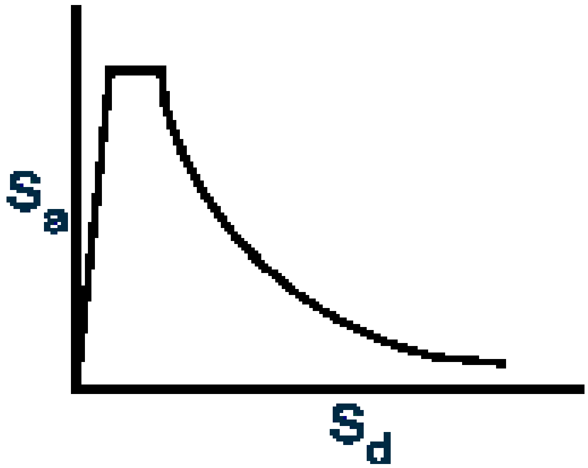
The Capacity Spectrum Method's goal is to establish suitable demand and capacity spectra for the system and to determine its intersection point. During this process, performance of each structural component is also evaluated. The spectrum of capacity is obtained by converting the base shear versus the spectrum of roof displacement into a spectral acceleration versus the spectral displacement as shown in Fig1.3(a). The intersection between a corresponding demand curve and the capacity curve is called the performance point.Capacity curve, in terms of base shear and roof displacement, is converted to capacity spectrum, which is a representation of the capacity curve in Acceleration Displacement Response Spectra (ADRS) format (i.e., Sa versus Sd) as shown in Fig1.3(b). This curve is obtained by redrawing the design earthquake response spectra as a curve of spectral acceleration v/s spectral displacement as shown in Fig 1.3(c).



(a)



(b)



(C)

Fig 1. 3Curves in capacity spectrum method: (a) Roof deflection, Δroof, plotted versus base shear, V; (b) Spectral displacement, Sd plotted versus spectral acceleration, Sa; (c) Response spectrum

1.7 PERFORMANCE PARAMETERS

Performance level of structures against earthquakes describes limiting damage condition that assumed to be satisfactory for a given building and a given ground motion.Moreover, building damages, danger to life safety of occupants in the building due to the damage, and post-earthquake serviceability of the building describe and control the limiting damage condition.Added to that, building performance level against earthquakes is a combination of the performance of both structural and nonstructural components.Lastly, performance levels of building structures against earthquake will be presented in the following sections.

Fig 1. 4Performance Level Parameters

Performance levels of buildings against earthquakes are as follows:

- Immediate occupancy performance level (IO)
- Life safety performance level (LS)
- Collapse prevention performance level. (CP)

Fig 1. 5Immediate Occupancy (IO) Performance Level

Immediate occupancy performance level

- The structure experience light damages
- There is no permanent drift.
- The building retains original strength and stiffness substantially.
- Minor cracking of facades, partitions, and ceilings as well as structural elements.
- Elevators can be restarted.
- Fire protection operable.
- The building space and systems are anticipated to be fairly usable. however, equipment and contents are generally secure but may not operate due to mechanical failure or lack of utilities.
- Concrete frame experience minor hairline cracking, limited yielding at few locations, and no crashing (strain of concrete less than 0.003)
- Steel moment frames experience minor local yielding at few locations. No buckling, fracture, and observable distortion of members.
- Lastly, braces of Braced steel frame structure suffer minor yielding or distortion

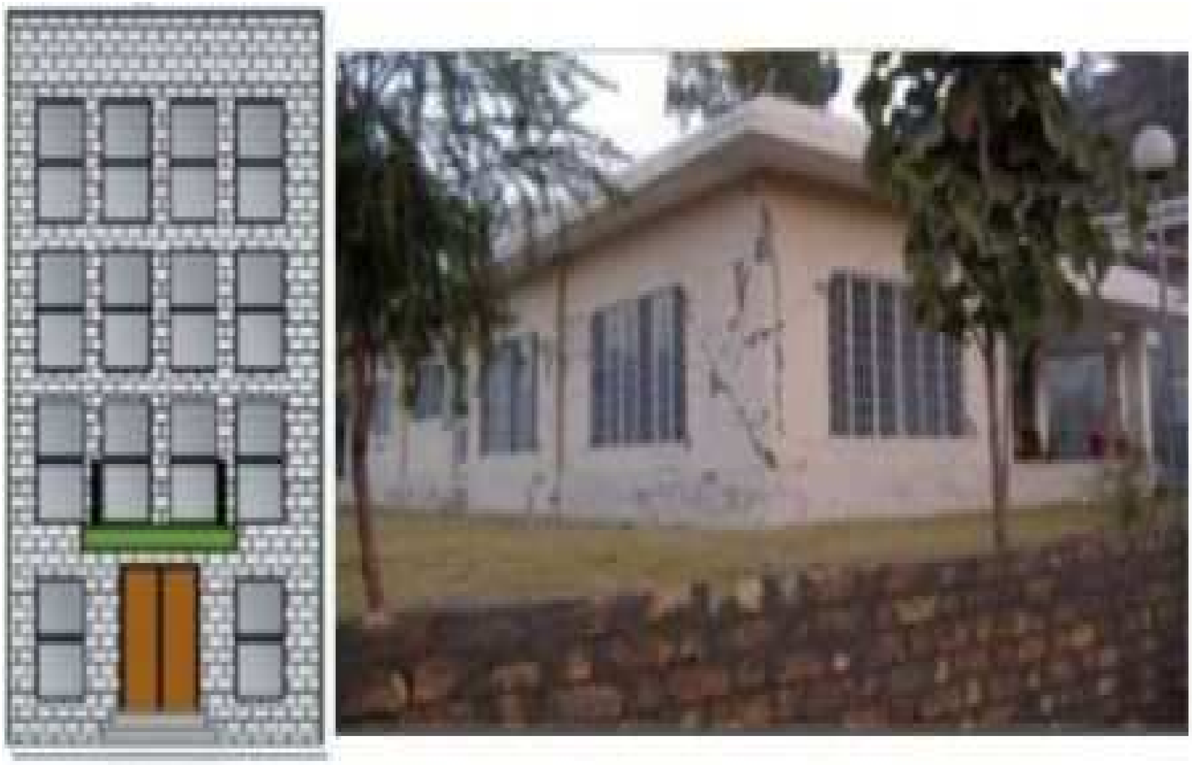


Fig 1. 6Life Safety (LS) Performance Level

Life safety performance level

- This level intended to obtain a damage condition that presents a substantially low probability of danger to life safety. Whether the danger is due to structure damage or fallen of nonstructural components of a building.
- The building experiences moderate overall damage
- All stories of a structure retain some residual strength and stiffness left in.
- Gravity-load bearing elements function.
- There will be no out of plane failure of walls or tipping of parapets.
- However, the structure undergoes some permanent drift.
- Partitions suffer damage.
- Building may be beyond economical repair.
- Falling hazards mitigated but many architectural, mechanical, and electrical systems are damaged.
- Concrete frame beams damage extensively, shear cracking and cover spall off occur in ductile columns, and minor cracking develops in no ductile columns.
- Hinges create in steel moment frames. In addition to local buckling of some beams, serious joint distortion, and fracture of isolated moment connection. However, shear connection would remain sound and few elements might suffer partial fracture.
- Lastly, in braced frames, majority of bracing buckle or yield but do not fail entirely, and several connections may fail as well.

Fig 1. 7Collapse Prevention (CP) Performance Level

Collapse prevention performance level

- This level of building performance mainly relates to the vertical load carrying system and the structure need to stable under vertical loads only.
- Generally, the building damage is severe
- The structure retains little residual stiffness and strength.
- However, load bearing columns and walls function.
- The building suffers large permanent drifts.
- Some exits blocked.
- Infill and un-braced parapets failed or at incipient failure.
- Building is near collapse.
- Nonstructural components damage extensively.
- In concrete frames, hinges and extensive cracking develop in ductile elements, no ductile columns experience splice failure and limited cracking, and short columns damage seriously.
- Beams and columns distort heavily in steel frames. Added to that, several moment connections fracture but shear connections remain intact.
- Finally, braces yield and buckle extensively in braced frames, and even many of them along with their connections could fail.

Performance point (PP): Indicates the damage state for which building is to be designed. The displacement at (pp) is the target displacement (Δt) also called design displacement (Δd). To know the performance of the building we need to know the performance point.

- **If$\Delta pp<\Delta IO$, it implies IO building.**
- **$\Delta pp>\Delta IO\&<\Delta LS$, LS building.**
- **$\Delta pp>\Delta LS\&<\Delta CP$, CP building**

Performance Levels

The selection of performance criteria acceptable to all the concerned parties is essential. Three performance levels are now under consideration as recommended by FEMA (2000a) for the seismic risk evaluation of steel structures. They are collapse prevention (CP), life safety, and immediate occupancy (IO) of a structure. Collapse prevention represents a performance level of serious structural damage that may lead to collapse. Obviously, a structure wills Performance Levels The selection of performance criteria acceptable to all the concerned parties is essential. Three performance levels are now under consideration as recommended by FEMA (2000a) for the seismic risk evaluation of steel structures. They are collapse prevention (CP), life safety, and immediate occupancy (IO) of a structure. Collapse prevention represents a performance level of serious structural damage that may lead to collapse. Obviously, a structure will

Table 1. 2Structural Correlation of Performance Levels, Probability of Exceedance Return Period and Allowable Drift

Performance level	Probability of exceedance	Earthquake return period	Allowable drift (δallow)
CP	2% in 50 years	2,475-year	0.050 *H
LS	10% in 50 years	475-year	0.025*H
IO	50% in 50 years	72-year	0.007*H

be unusable at this damage level. Life safety is a state of considerable structural damage; some structural members may fail, and the structure must be repaired before reoccupancy. The IO performance level is characterized by a practically undamaged structure, so the structure can be used immediately. FEMA (2000a) proposed the performance levels of CP, LS, and IO in terms of the probability of exceedance, earthquake return period, and allowable overall lateral and/or interstory drifts (Table1.2). The term H in the Table 1 represents the total and story height of the building to consider the overall and interstory drift, respectively, for the two LSFs. This study uses the allowable drift values in Table1.2 to document the implementation of PBSD.

1.8 OBJECTIVES

1 To formulate FEM model for frame analysis.
2 To perform performance based seismic design (PBSD) of steel frames
3 To obtain preliminary results for frame analysis to get performance parameter.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

In this chapter it would be difficult to include a comprehensive analysis of the literature relating to the modelling of systems in their entirety. This segment is a short summary of previous research on applying the PBSD study specially pushover analysis of steel frames. This study of the literature focuses on recent contributions relating to pushover analysis of steel frames and previous activities most closely linked to the needs of the present research.

Camacho, et. al. [1] developed afinite element FE model to analyze the behavior of steel frames by performance based seismic design using rigidities of connections. For the information on reliability, risk or the probability of failure in steel structure Monte Carlo simulations are used. For the implications of rigidity of PR connections in steel buildings stress based FE algorithm is formulated. The accuracy and efficiency of model is analyzed by taking three examples of steel frame. First verification is done on two storey frame. The robustness and implementation potential of the PBSD approach were demonstrated with the help of 9 story and 20 story steel frame. They evaluated reliabilities of steel frame structure for the overall and interstory drift.The performance criterions were successfully verified by PBSD which made believes that post-Northridge design requirement increase the possibility of achieving more-resilient and damage tolerant steel structure.

Papadopoulos, et. al. [2] analyzed vulnerability objectives of structure by performance based seismic design (PBSD). From the test cases examined it was observed that although PBD with vulnerability constraints is up to 10% more expensive compared to the standard PBD designs in case of initial cost, it is 20-25% cheaper with reference to the life cycle cost. By targeting limit state probabilities of exceedance vulnerability objectives are introduced. This is achieved by performing additional probabilistic design check. A structural optimization problem is considered in order to assess the designs obtained using the proposed approach with respect to standard PBSD procedure with deterministic constraints. For the optimum design of 3D RC building two procedures have been applied. It has been demonstrated that the concept of PBD using vulnerability constraint can be easily integrated into a structural optimum design that fulfill the provision of modern framework for seismic design of structures.

El-Zanaty, et. al. [3] analyzed the nonlinearity of steel frames by finite element methods (FEM). In these nonlinear methods of framed analysis based on large deformation theories, applicable to both elastic and inelastic solutions of plane frame problems were presented. In the inelastic formulation, the effect of axial loads on the stiffness of the structure is considered.The effect of axial loads on the stiffness of the structure is considered.The theory is based on the simple geometric approximationwhichpermits the virtual work equations to be derived in a manner consistent with the full nonlinear strain displacement equations without introducing further approximations.When this geometrically nonlinear theory is combined with Shanley's tangent modulus concept (15), and the incremental Newton-Raphson equations are formed by the finite element method, a numerical technique emerges which is capable of solving inelastic frame stability problems for planar frames of arbitrary geometry.

Salajegheh, et. al [4] has presented a performance based design of steel frames.Analytical researches on the proposed beam to column connections have proved that this connection can be assumed rigid. This research describes the type of rigid beam to column connection and discussed its moment curvature behavior obtained from finite element analysis using ABAQUS software. Estimation of acceptance criteria required for performance based seismic design of such frames by using nonlinear static analyses results were developed.Ant Colony Optimization (ACO) theory was employed to accomplish the performance based design process, utilizing a MATLAB program for optimization and OpenSEES (Open System for Earthquake Engineering Simulation) software capabilities in structural analysis.

Mehrabian, et. al. [5] has presented a nonlinear analysis of steel frames. Northridge earthquake in 1994 caused a catastrophic damage to the steel structure which is constructed by an earthquake resisting building design criterion. The beam column connection was PR type which caused the nonlinearity in structure. In this, a case study of analytical investigation of nonlinear seismic performance of a 9-story steel frame was presented. Fourparameter Richard model, a mathematical model was proposed first to represent moment–relative rotation (M–θ) curves fora proprietarysteelconnection.Themodelispresentedand described was elsewhere (4)andcangenerate(M–θ)curvesfor otherbeam–columnassemblies. Bolted-web, welded-flange connections withadequateductility (BWWF-AD) connections used in this study have large elastic stiffness, ductility, and energy absorption capacity in comparison with other types of PR connections.Whengroundshakingwassignificantenough toproducelargelateraldisplacements, thepresenceofBWWF-ADconnectionsimprovedtheresponsesoftheframes.

Reyes-Salazar, et. al. [6] analyzed nonlinear seismic response of steel structures with semi-rigid and composite connections.Steel frames are usually analyzed assuming all the connections are fully restrained, but experiments suggests that its rarely true.This practice introduces unintended flexibility in the frame which causes the damage to structure during earthquake.For these studies, nonlinear time domain seismic analysis algorithm developed, three steel frames are excited by 13 earthquake time histories. Twelve of them were recorded during the Northridge earthquake of 1994.One of these 12 earthquake time histories can be used to represent the Northridge earthquake in future designs.The nonlinear seismic responses of steel frames with fully restrained, partially restrained and composite connections are evaluated and compared in terms of the maximum interstory and maximum top lateral displacements.To define the rigidity of a connection, a parameter called the T ratio is introduced. Initially, the T ratio of all the connections is assumed to be 0.9, making them fully restrained. The results indicate that this assumption is inappropriate and gives unconservative responses in most cases.Further parametric study indicates that, at least for seismic analysis, PR or composite connections should be designed for a T ratio as close to 1 as possible to represent an FR connection. Otherwise, the lateral displacement failure criterion should also be checked for less than ideal FR connection conditions.

Foley CM. [7] a review of current state-of-the-art seismic performance-based design procedures and provided the vision for PBD optimization development. It is acknowledged that the development of optimized PBD procedures for seismic structure engineering is urgently needed.

R. Hasan, et. al. [8] conducted a simple computer-based push-over analysis technique for performance-based design of earthquake-loadable building frameworks. And found that for push-over analysis, rigidity-factor for elastic analysis of semi-rigid frames and the stiffness properties for semi-rigid analysis are taken directly.

B. AKBAS,et. al. [9] conducted a push over analysis on steel frames to estimate the seismic demands at different performance levels, which requires the consideration of inelastic behavior of the structure.

X.-K. Zou et al., [10] presented a successful technique combining Pushover Analysis and numerical optimization techniques to automate the Pushover drift output design of reinforced concrete structures. PBD using nonlinear pushover analysis, which typically requires repetitive computational effort, is a highly iterative method required to fulfill the requirements of code.

Oğuz, et. al. [11] Used in pushover analysis to predict the action imposed on the structure due to randomly chosen individual ground motions causing elastic deformation by observing different rates of nonlinear response. Of this reason, pushover analyzes were performed on reinforced concrete and steel moment resistant frames covering a wide range of fundamental periods using various invariant lateral load patterns and Modal Pushover Analysis. On frame structures, the precision of estimated Procedures used to estimate target displacement was also studied. DRAIN-2DX and SAP2000 rendered pushover analyzes.Pushover analyses were performed by both DRAIN-2DX and SAP2000. The primary observations from the study showed that the accuracy of the pushover results depended strongly on the load path, the characteristics of the ground motion and the properties of the structure.

Mehmet et al., [12] Explained that the structural engineering profession was using a nonlinear static method or pushover analysis because of the simplicity of Push over Analysis. Pushover research is performed on the basis of the FEMA-356 and ATC-40 guidelines for various nonlinear hinge properties present in some programs and he pointed out that plastic hinge length (Lp) has significant effects on the frame's displacement capability. The default-hinge properties (Program Default) cannot take proper account of the orientation and axial load level of the columns.

Shuraim et al., [13] In order to test its applicability, the non-linear static analytical technique (Pushover) implemented by ATC-40 was used to analyze the current configuration of a building. He performed nonlinear pushover analysis shows that the frame is capable of withstanding the pre-assumed seismic force at all beams and columns, with some important yield.

Girgin. et al., [14] pushover analysis has been the preferred approach by the major rehabilitation guidelines and codes for seismic performance assessment of structures because it is computationally and conceptually simple. Pushover analysis enables the sequence of yield and failure to be tracked at member and structural level, as well as the development of the structure's overall capability curve.

A. Shuraim et al., [15] the non-linear static analytical technique (Pushover) introduced by ATC-40 was used to test the current reinforced concrete frame configuration. The pushover approaches estimated possible structural defects in the reinforced concrete frame when exposed to a mild seismic loading. In this method the design was evaluated by redesigning under selected seismic combination to show which members would require more strengthening. Most columns needed substantial additional strengthening, suggesting their weakness when experiencing seismic forces.

Athanassiadou [16] Two ten-storied two-dimensional plane stepped frames and one ten-storeyed standard frame built for high and medium ductility classes according to Euro code 8 (2004) were analyzed. This work validates the design methodology for irregular buildings which requires linear dynamic analysis recommended in Euro code 8. The stepped buildings, designed according to Euro code 8 (2004), were found to behave satisfactorily under the earthquake design basis and even under the maximum considered earthquake (involving ground motion twice as powerful as the earthquake design basis). In the case of the "collapse protection" earthquake, inter-storey drift ratios of irregular frames were found to remain very small.This reality, combined with the restricted structure of plastic hinge in columns, excludes the possibility of creating a collapse mechanism in the irregularities neighborhood. During the design basis earthquake, plastic hinge formation in columns is seen to be very minimal, occurring only at locations not prohibited by the code, i.e. at the base and top of the structure. It was concluded that the capacity design procedure given by Euro code 8 is fully effective and can be characterized by conservatism, especially in the case of the design of columns with high ductility.The over-strength of the abnormal frames is close to that of the normal frames, with the over-strength ratio values for medium – high ductility ranges being 1.50 to 2.00. The author reported pushover analysis findings using "uniform" load pattern as well as a "modal" load pattern that accounts for multimodal elastic analysis findings.

A.Kadid and A. Boumrkik [17] proposed use of Pushover Analysis as a viable method for assessing the vulnerability of an Algerian-designed building to damage. The Pushover analysis was a series of incremental static analyzes performed to develop a building capacity curve. A target displacement was calculated based on capacity curve which was an estimation of the displacement that the earthquake design would generate on the building. The structure encountered at this target displacement is considered to be indicative of the damage sustained by the building while experiencing ground shaking design phase.Since the behavior of reinforced concrete structures under seismic loads could be highly inelastic, plastic yielding effects would dominate the global inelastic performance of RC structures and consequently the accuracy of the pushover analysis would be influenced by the capability of the Analytical models to capture these effects.

P.Poluraju and P.V.S.N.Rao [18] by conducting Push over Analysis, the behavior of framed buildings was studied, most of the buildings collapsed were found to be deficient in meeting the requirements of the current codes. Then G+3 building was modeled and analyzed, the results obtained from the study show that under seismic loads the properly designed frame will perform well.

NarenderBodige, Pradeep Kumar Ramancharla [19] modeled a 4-story building with 1 x 1 bay 2D using AEM (Applied Element method). AEM is a discrete system in which the elements are bound by pairs of normal and shear springs that are distributed along the edges of the elements and each pair of springs completely represents stresses and automatically shapes deformation and position of plastic hinges. Gravity and lateral loads were added to the structure as per IS 1893-2002 and constructed using IS 456 and IS 13920. In both cases, the displacement control pushover analysis was performed, and the pushover curves were compared. It was found as an observation that AEM gave good curve of capacity for representation.From the case studies it was found that when ductile detailing was adopted, the building capacity significantly increased. Effect on concrete grade and steel was also found not to be highly significant.

2.2 LIMITATIONS OF EXISTING STUDIES

Many researchers have performed many experimental and theoretical works in the area of the pushover study of the steel frames. Nowadays the idea of pushover analysis is increasing rapidly.

This work is about the steel frame pushover study. In previous research, the applications of pushover analysis of the steel frames were extensively studied. However, several researchers conducted on the pushover analysis experimentally and analytically but the study of pushover analysis is done with minimal work.

2.3 CLOSURE

The literature review has suggested that use of a pushover analysis of the steel frame is feasible. So it has been decided to use STAAD Pro Advanced for the modeling and analysis. With the help of this software study of steel frame has been done.

CHAPTER 3

METHODOLOGY

3.1 MODELING METHODOLOGY:

3.1.1 INTRODUCTION

The structural system is modeled as:

1. **2D Moment Resisting Frame**
2. **3D Concentric Braced Frames**
   a. **Steel Frame Without Bracing**
   b. **Steel Frame with External Bracing**
   c. **Steel Frame with Optimum Bracing**

All above frames are analyzed by Nonlinear Pushover Analysis.The FEM based structural software used for modeling and analysis of the steel frame is STAAD Pro. Advanced software package.

3.1.2 MODELLING

The basic approach for using the program is very straightforward. The user establishes grid lines or can directly create frame from structural wizards,defines material, type of support and section properties graphical user interface. All the types of loads that the structure is subjected to, can bedefined and assigned to the appropriate structural components. Nonlinear Pushover Analysis properties like gravity loads, convergence criteria for geometric nonlinearity, spectral parameters like critical damping and mapped spectral acceleration, hinge properties (FEMA) can be defined. Finally, the nonlinear static analysiscan be performed and the results are generated in graphical or tabular form that can be printed toa printer or to a file for use in other programs. The following topics describe some of theimportant areas in the modeling.

3.1.3BASICSTEPSIN STAAD PROADVANCEDWHILEPERFORMINGPUSHOVERANALYSIS

1) **Structure Wizard:** Used to parametrically generate a structural model and then transfer and superimpose it on the current structure in STAAD.Pro. Opens when Geometry > Run Structure Wizard is selected in the STAAD.Pro window.

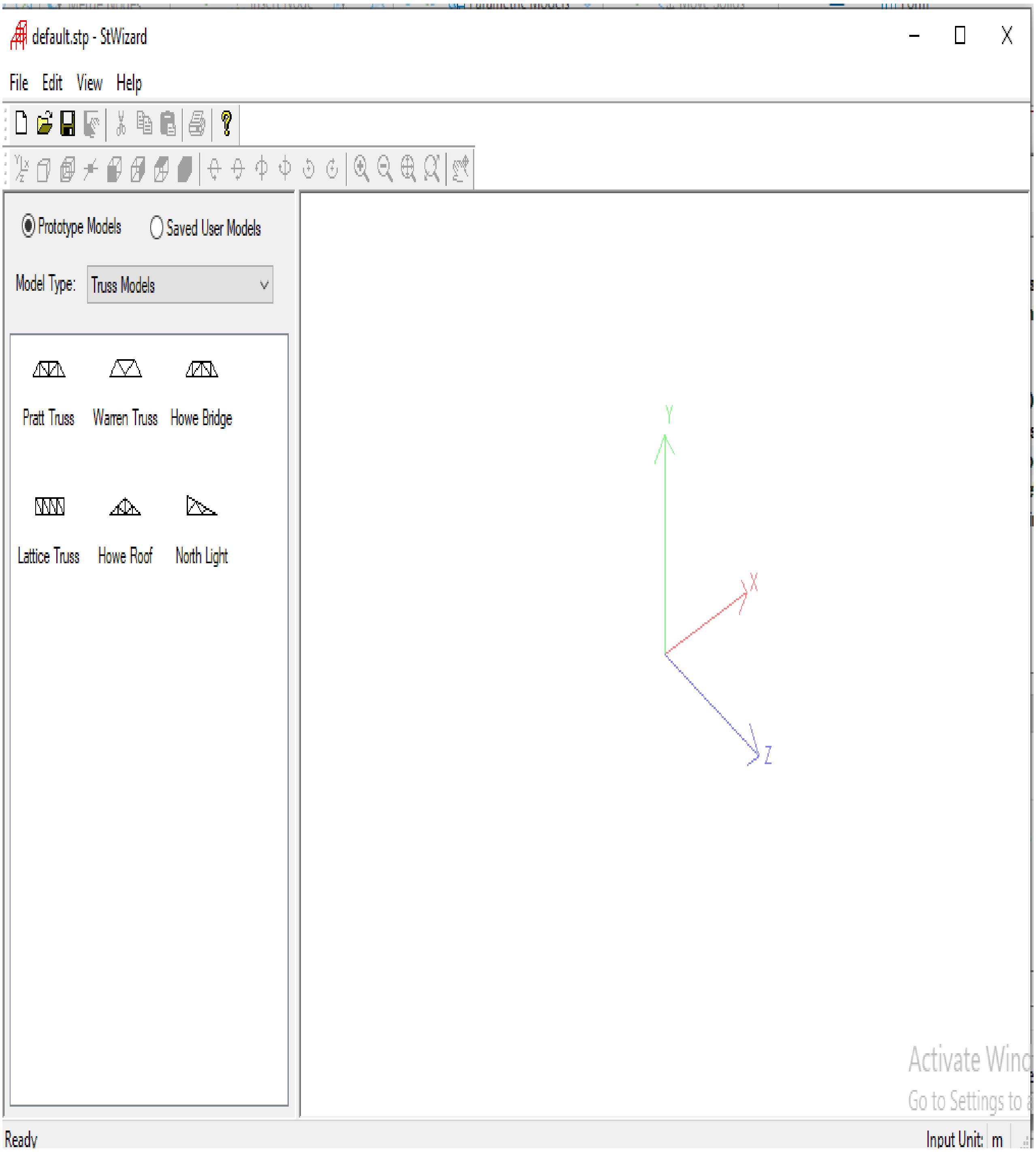


Fig 3. 1Structure Wizard Window

The Prototype Modelsshown in fig.3.1and Saved User Models options appear on the top of the left side of the screen. If the Prototype Models option is selected, the Model Type will list the types of prototype structures available (such as Trusses, Frames, Plates, Solids, etc.) as shown below. If the Saved User Models option is selected, the Model Type will display the list previously done and saved models by the user. As steel frame need to be created, portal frame option is selected

After that prototype model window will displays different types of frame models such as bay frame, cylindrical frame, grid frame and circular beam, etc. Here bay frame option is selected for further modeling and parameters can be given as shown below in fig 3.2

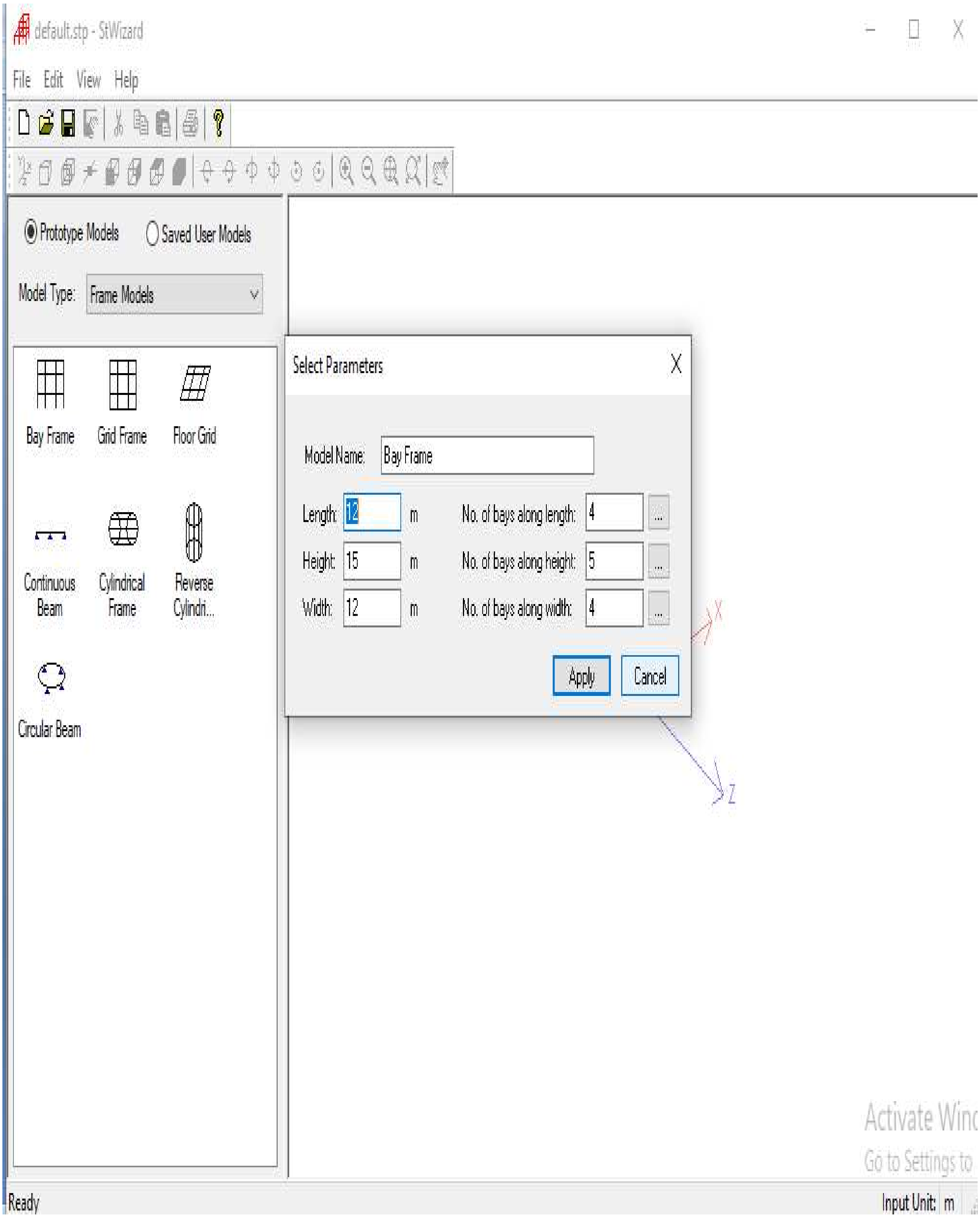


Fig 3. 2Bay Frame Parameters Window

Once the parameters are defined to the bay frame, from file tab merge model to the STAAD Pro. is selected.

2) **Specifying Member Properties:** In this procedure, cross section properties to the beam and column can be assigned. For this properties page in analytical modeling control is selected. The properties-whole structuredialogue opens as shown in fig.3.3

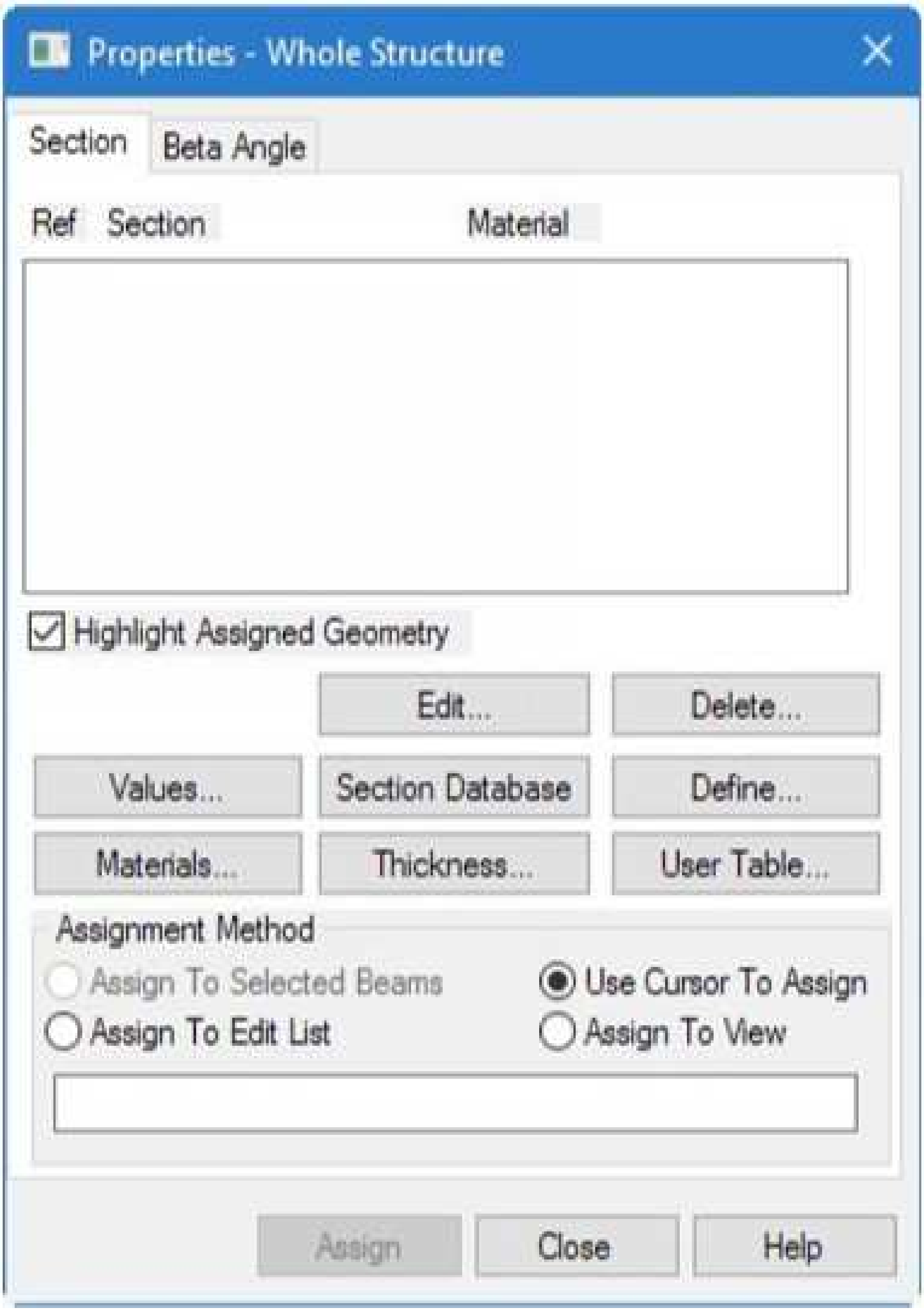


Fig 3. 3Member Properties Window

After that, Section database is clicked and Section Profile Tables dialogue box opens as shown in fig.3.4

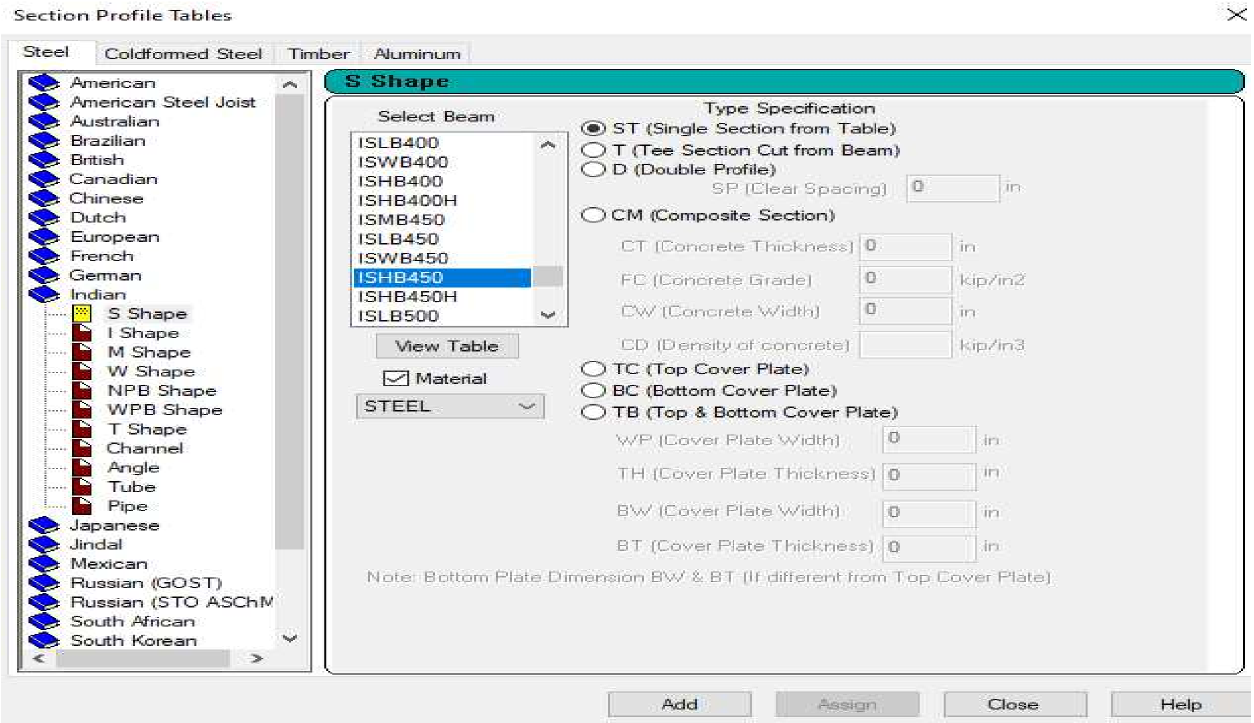


Fig 3. 4Section Profile Tables

Select the S Shape from Indian option. The property type we wish to create is S Shape from Indian Steel Table. ISHB 225 and ISHB 450 are selected as shown in below fig. 3.5

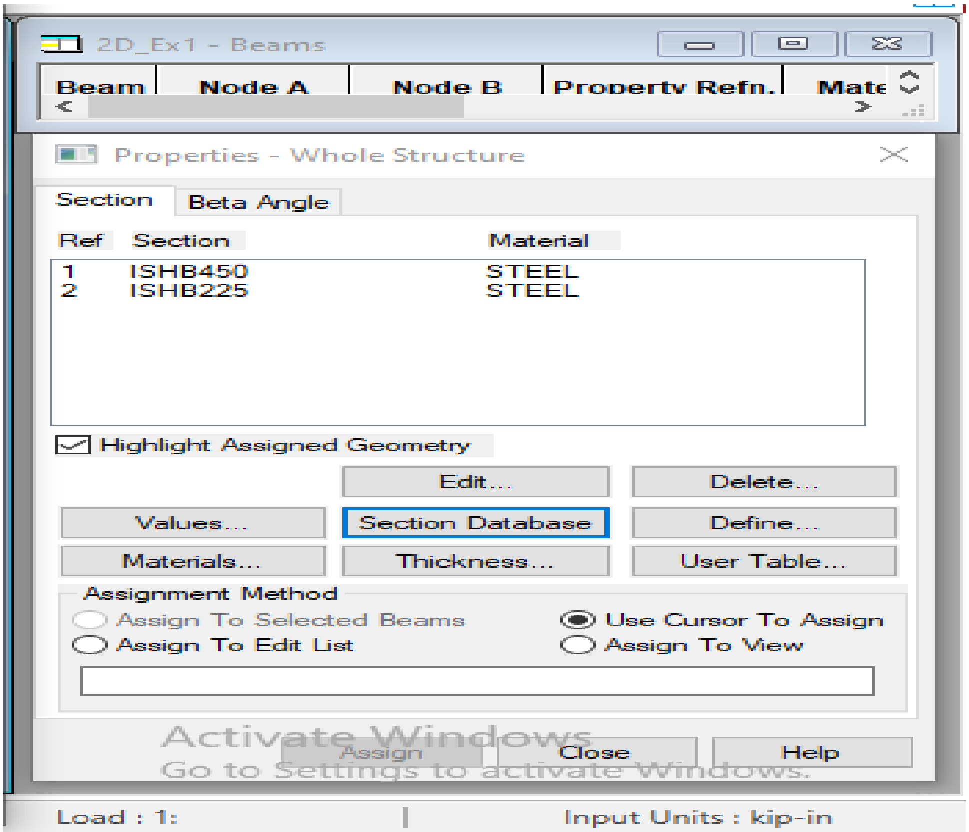


Fig 3. 5Cross Sections Window

After selecting steel cross sections from section database ISHB 450 section is assigned to the column and ISHB 225 is assigned to the beam, and for that following steps are followed.

a. Select the first property reference in the Properties dialog (ISHB450).
b. Select the Use Cursor to assign option in the Assignment Method group.
c. Click Assign.
d. Click on members 1 and 3.
e. To stop assigning properties, either:

Click Assigning
or
press the <Esc> key.

Repeat step 8 except to assign the second property reference (ISHB225) to member 2.

After the properties are assigned to the respective members, the member labels will indicate the section reference numbers as shown in fig. 3.6

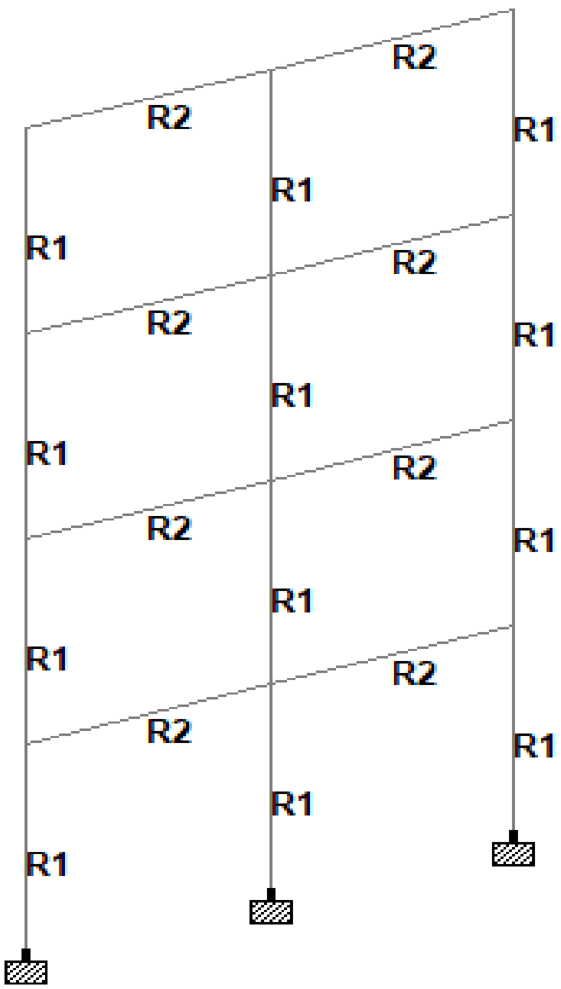


Fig 3. 6Section Property Reference no.

3) **Defining Material**: This command may be used to specify the material properties by material name. You will then assign the members and elements to this material name in the CONSTANTS command. Here the material used is steel as shown in fig. 3.7

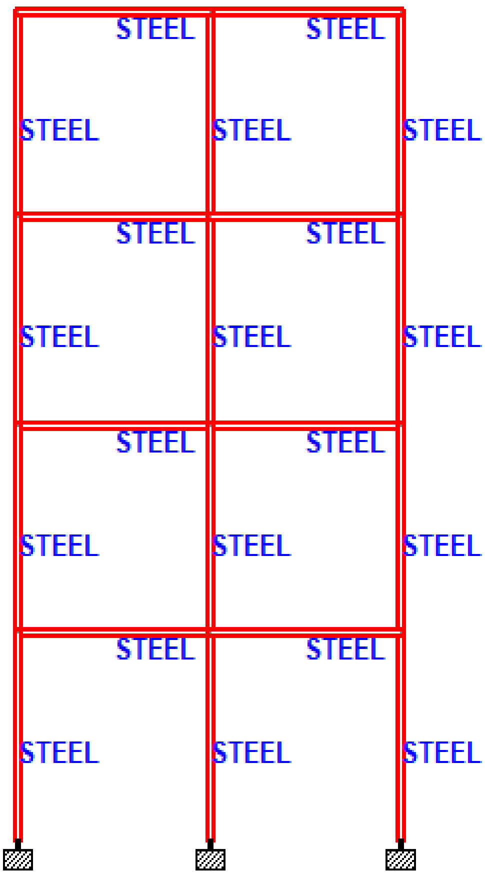
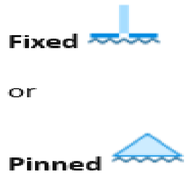


Fig 3. 7Material Properties

Table 3. 1Properties of Steel

Name	E kip/in2	Poisson's Ratio	Density kip/in3	Alpha /°F	Fy kip/in2	Fu kip/in2	Ry	Rt	Fcu kip/in2
ALUMINUM	10000.000	330E-3	98E-6	12.8E-6	0.000	0.000	0.000	0.000	0.000
CONCRETE	3150.000	170E-3	86.8E-6	5.5E-6	0.000	0.000	0.000	0.000	4.000
STAINLESSSTEEL	28000.000	300E-3	283E-6	9.9E-6	0.000	0.000	0.000	0.000	0.000
STEEL	29732.699	300E-3	283E-6	12E-6	0.000	0.000	1.000	1.000	0.000
STEEL_275_NMM2	29732.736	300E-3	283.7E-6	6.667E-6	39.885	59.465	1.500	1.200	0.000
STEEL_355_NMM2	29732.736	300E-3	283.7E-6	6.667E-6	51.488	68.168	1.500	1.200	0.000
STEEL_36_KSI	29000.000	300E-3	283E-6	6.5E-6	36.000	58.000	1.500	1.200	0.000
STEEL_50_KSI	29000.000	300E-3	283E-6	6.5E-6	50.000	62.000	1.500	1.200	0.000

4) **To assign fixed or pinned support**: To specify a node as either a fixed or pinned support, used the following procedure.
A fixed support is restrained against movement (translation and rotation) in all degrees of freedom.
A pinned support is restrained against translation only, but is otherwise free to rotate.
   a. Select the nodes that will have the same supports condition.
   b. On the **Specifications** ribbon tab, select one of the following tools in the supports group.



   c. The Create support dialog opens to the corresponding tab
   d. Either

Table 3. 2Support Dialogue

…	Do the following…
d the support type to the model and assign to the current node selection	iclick Assign.
d the support to the model for later assignment	click**Assign**.

The dialog closes.

5) **Load and Definitions:** Assigning load definitions and load cases is important part of every analysis. For this first click on loading command. The following tab of load definition will open as shown in fig.3.8

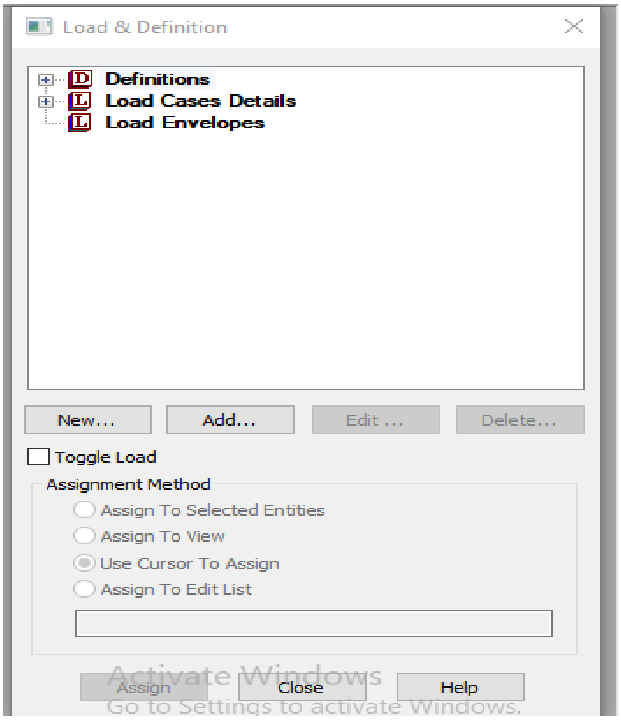


Fig 3. 8Loads and Definitions

For pushover analysis firstly we need to click on definitions then following tab will open showing various definitions as shown in fig. 3.9

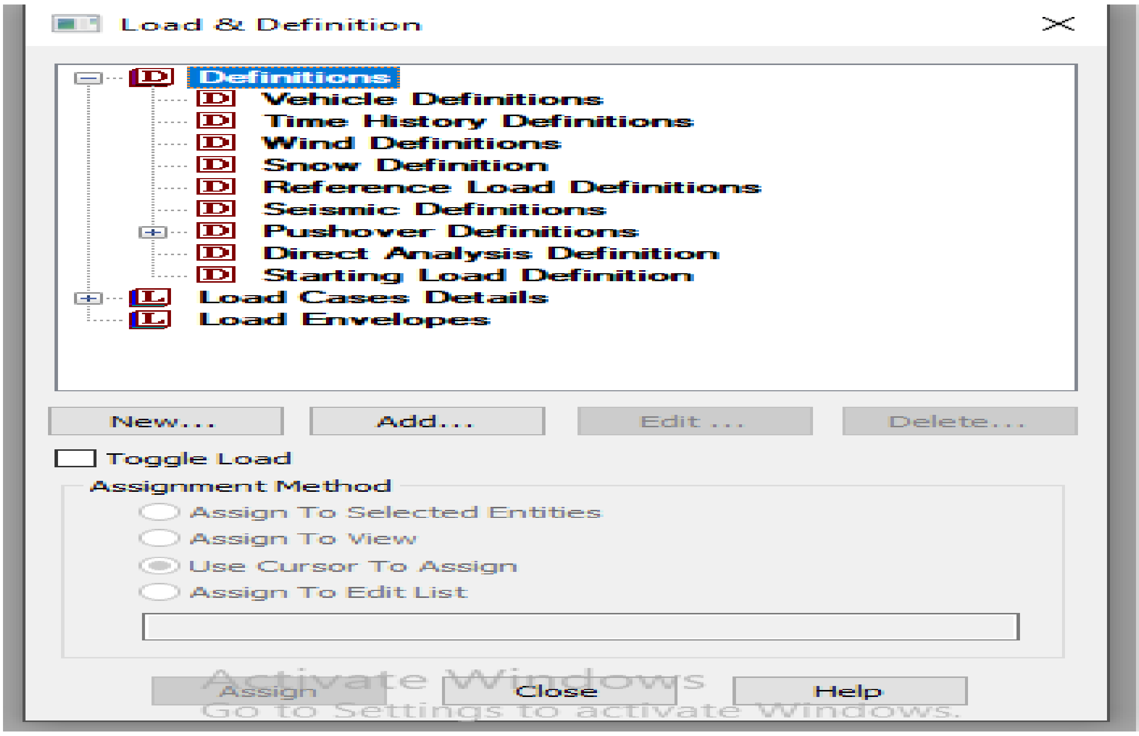


Fig 3. 9Pushover Load Definition

After selecting Pushover Definitions, click on add and following steps to be followed:

a. Define Input:

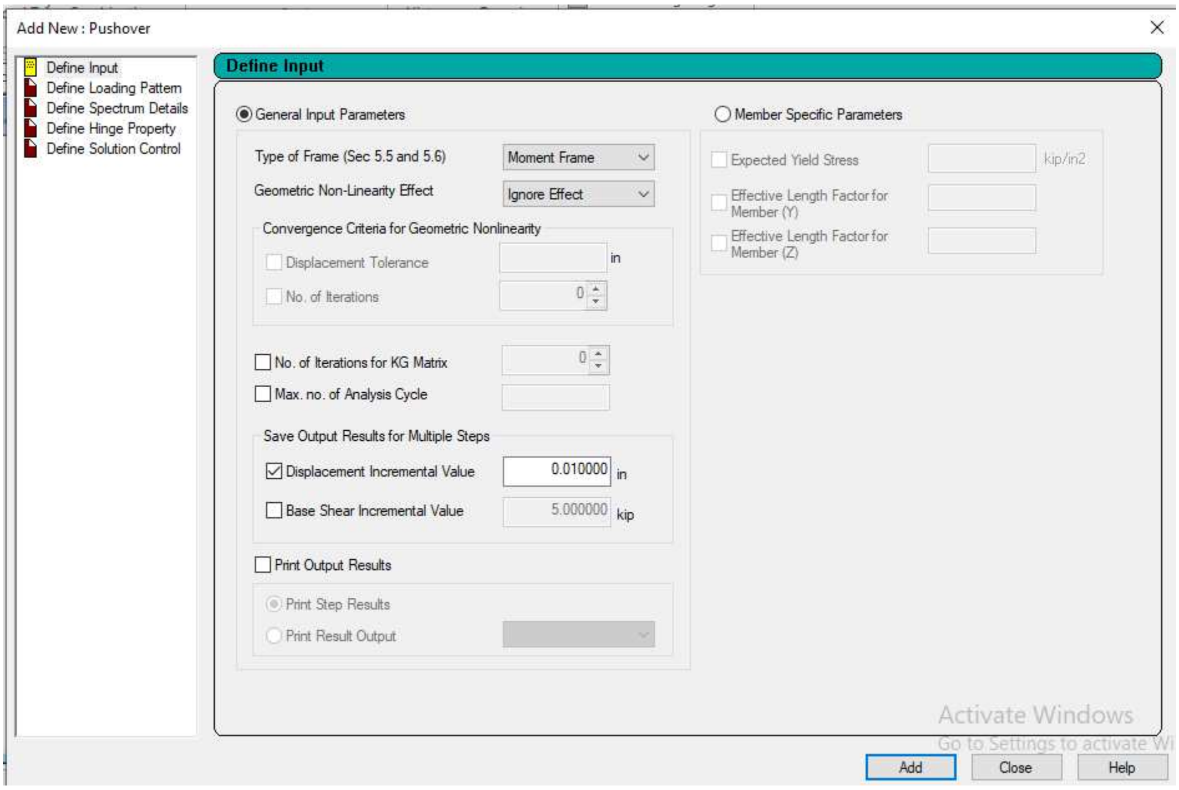


Fig 3. 10Define Pushover input

b. Defining Loading Pattern:

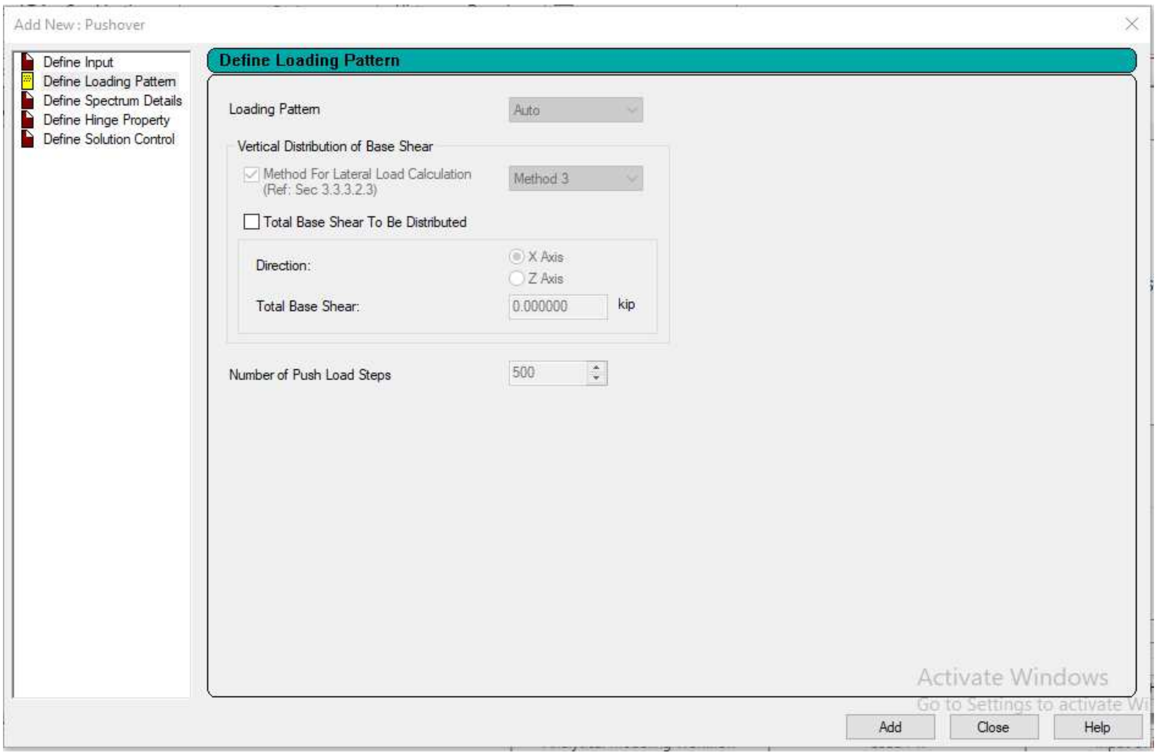


Fig 3. 11Defining Pushover Loading Pattern

c. Define Spectrum Details

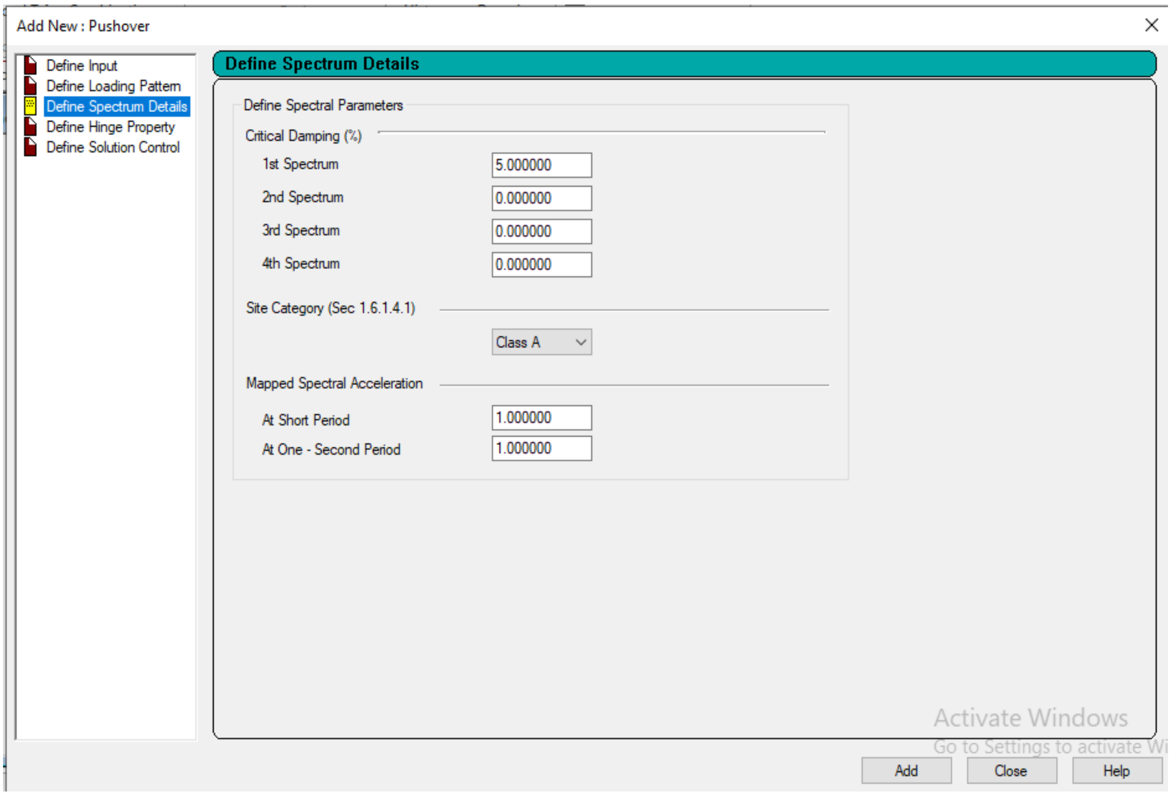


Fig 3. 12Defining Spectrum Details

d. Define Hinge Property

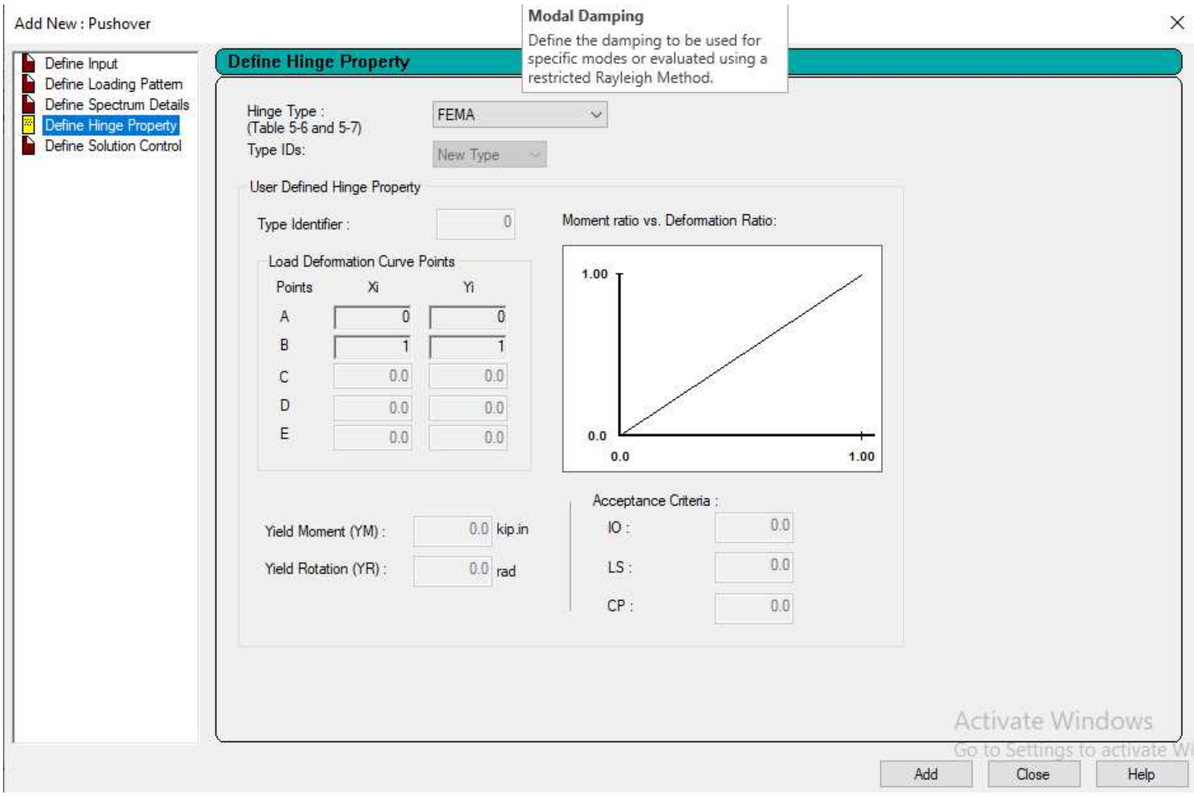


Fig 3. 13Defining Hinge Properties

e. Define Solution Control

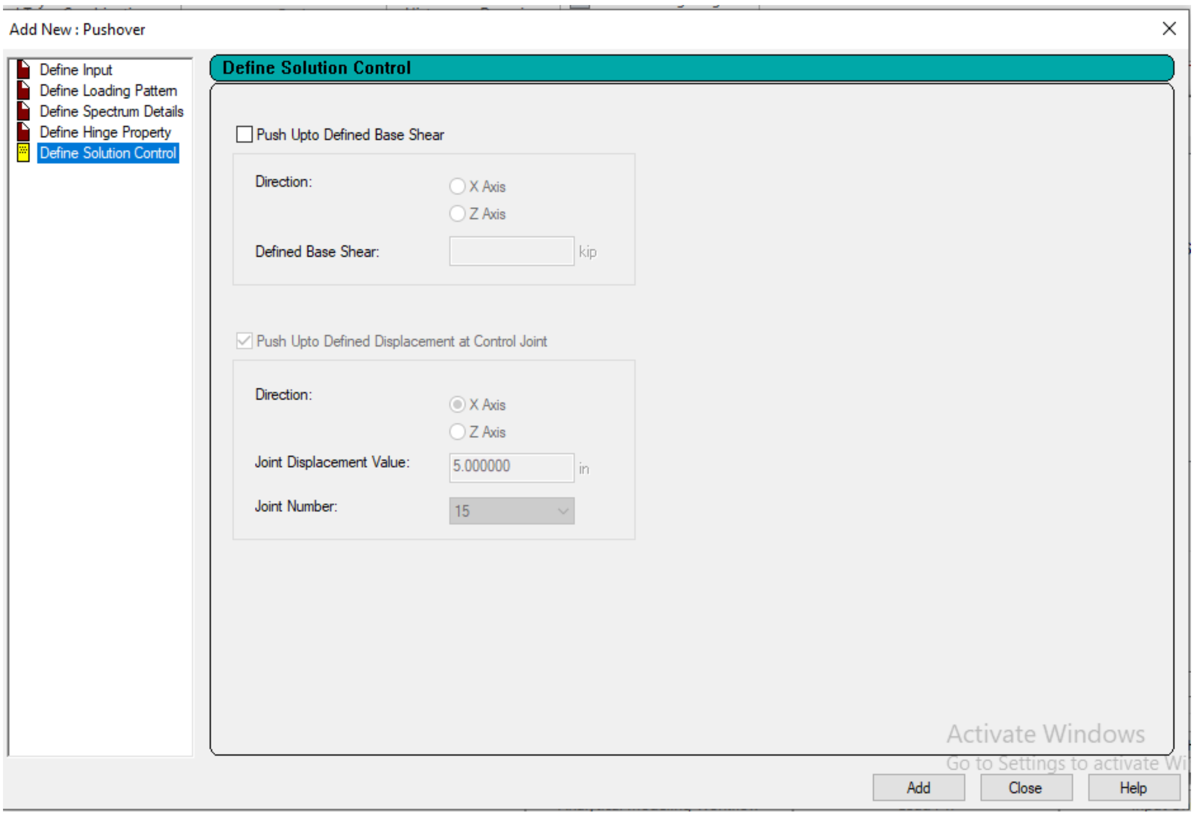


Fig 3. 14Defining Solution Control

f. Load Case Details: Under gravity Loading two load cases are defines. First one is self Wight of the steel frame as shown in fig. 18. It is assigned as -1 in global Y direction.

Second load case is of uniformly distribute load of 0.6 Kp/in in globay Y direction as shown in Fig. 22.

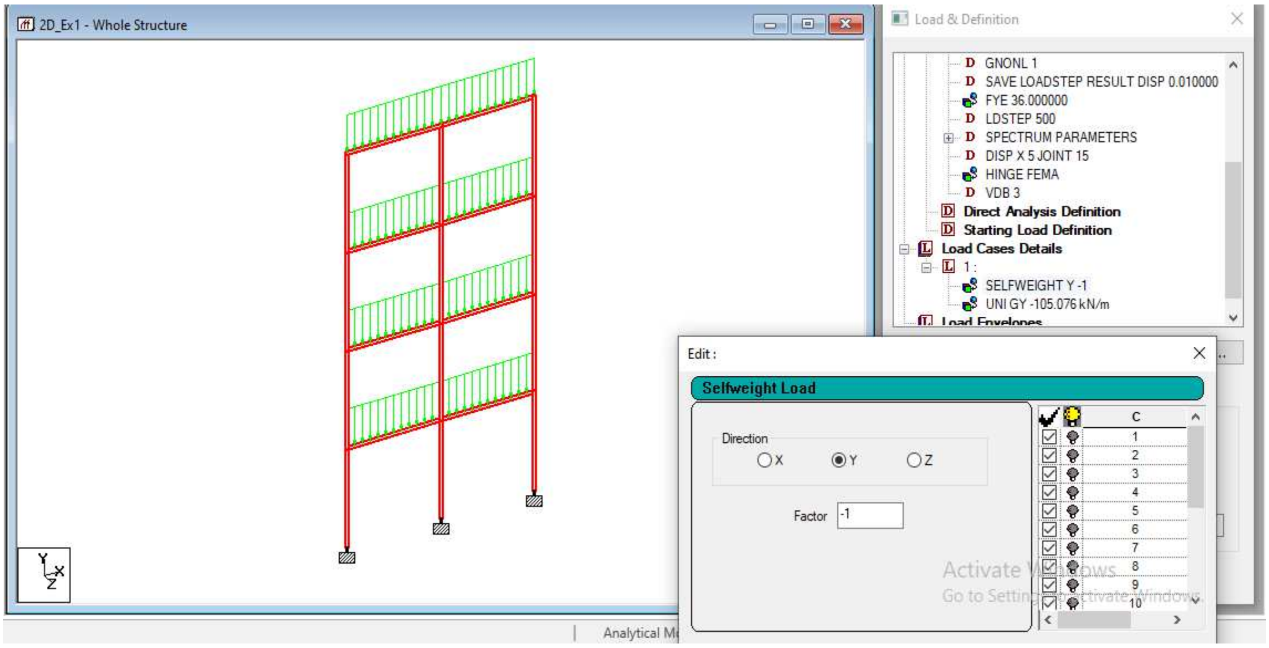


Fig 3. 15Load Case Details

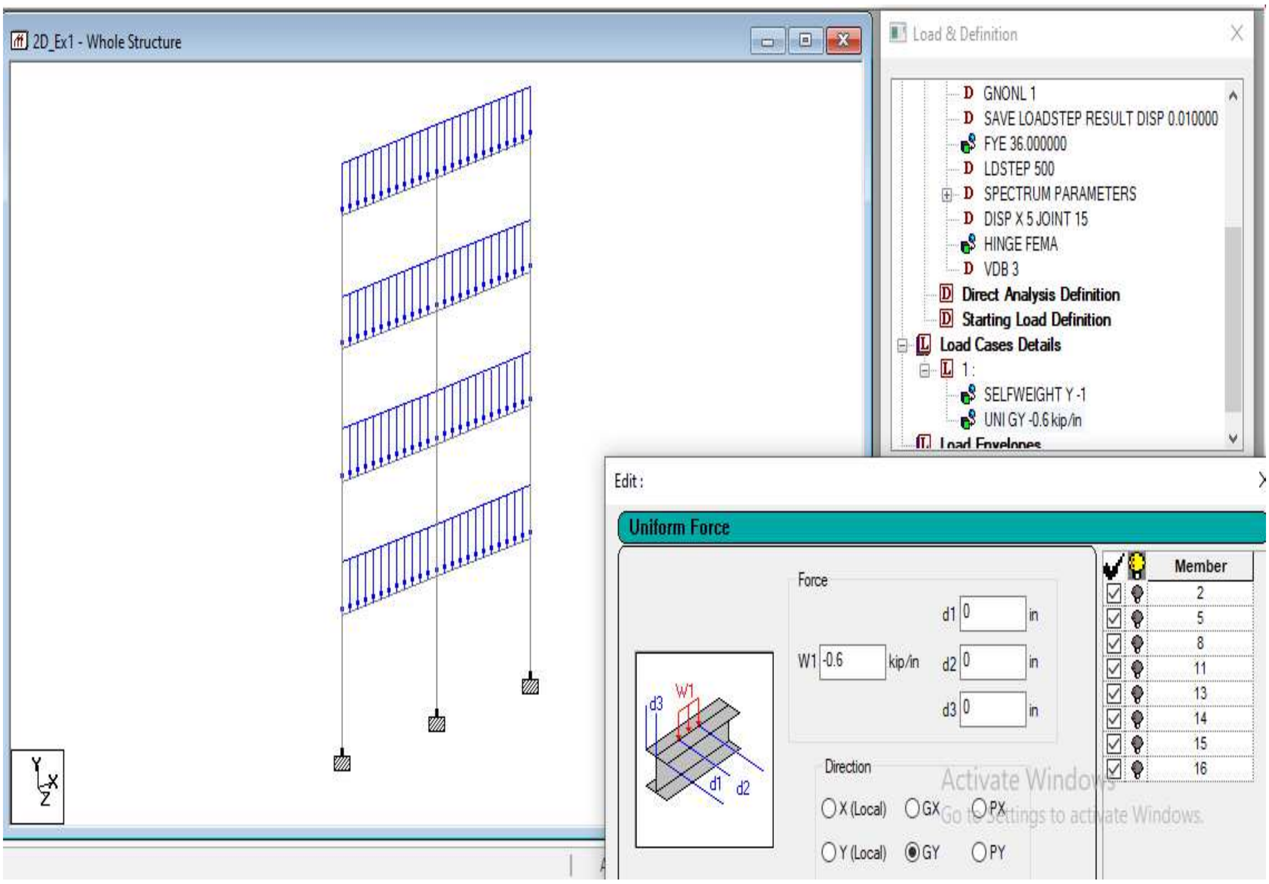


Fig 3. 16Uniformly Distributed Loads

6) **Perform Analysis:** To perform pushover analysis click on analysis > define command> click on pushover analysis> Add. These steps are supposed to follow while performing pushover analysis as shown in following Fig.3.17

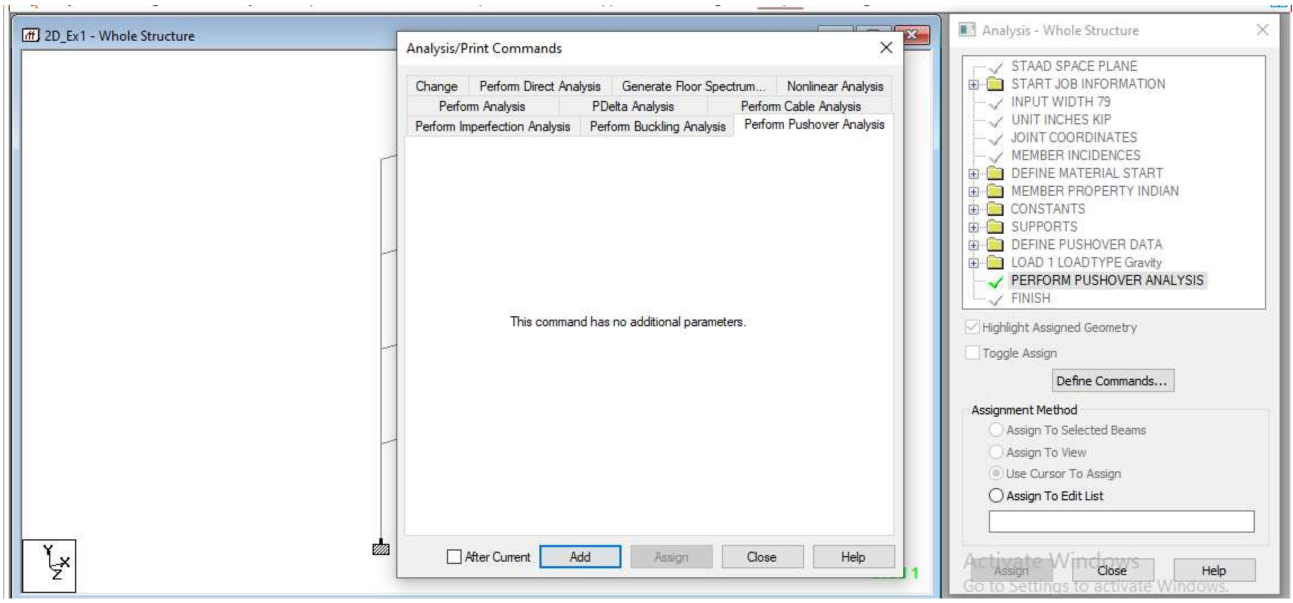


Fig 3. 17Performing Analysis

7) **Run Analysis:**Once you have completed the input file, use the following procedure to perform an analysis and optional design.

The STAAD analysis engine performs analysis and design sequentially with a single click. In order to carry out the design, these design parameters must be specified along with geometry, properties, etc. in the input file (this is referred to as a "batch" design). Also, note that you can change the design code used for design and code check before performing the analysis and design. The Analytical Modeling and Physical Modeling of the STAAD.Pro user interface are used to prepare the structural input data which is then passed to the STAAD analysis engine for general purpose structural analysis and design.

Either:

Select the Run Analysis tool in the analysis group on the analysis and design ribbon tag.

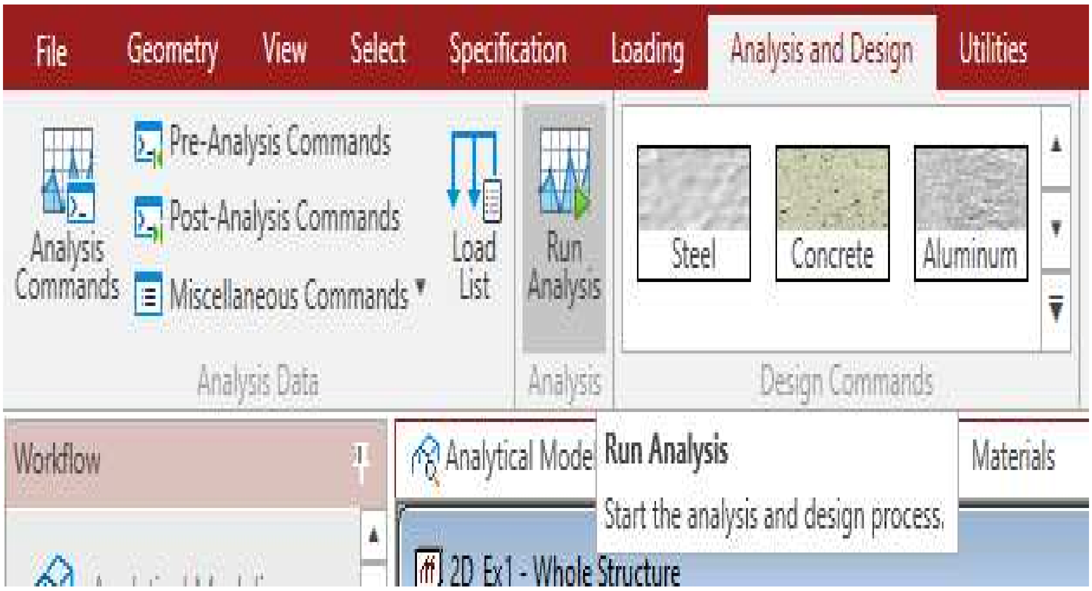


Fig 3. 18Run Analysis Tab

or

Press **<CTRL+F5>**

The STAAD analysis and Design dialog opens

During the analysis (and design, if specified), an output file is generated. This file may contain selected input data items, results and error messages. Optional print specifications can be used to include additional information in the output file.

1. Select an option for what action occurs when the dialog is closed:

   Open the output file

   or

   go to the post-processing mode

   or

   Remain in the analytical modeling mode

2. Click **Done**.

   (Optional) To review the output file is this option was not selected in the STAAD analysis and Design dialog, either:

   Select the **STAAD Output** tool in the **Utilities** group on the **Utilities** ribbon tab

or

Select the **View STAAD Output File** tool on the Quick Start toolbar.

Model: I > 2D Moment Resistant Steel Frame

First model considered for pushover analysis is of simple 2D, G+3 moment resistant steel frame as shown in following fig.3.19

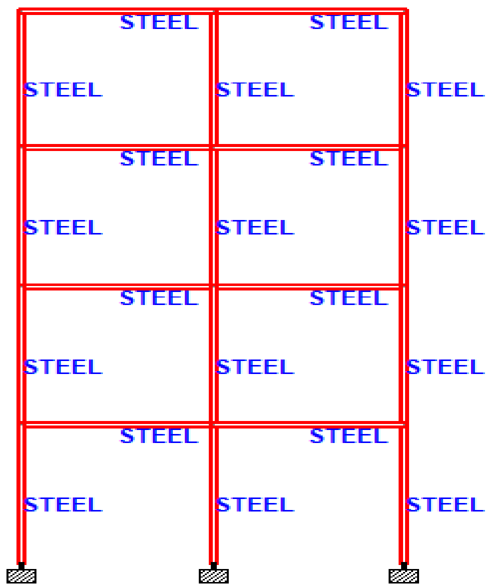


Fig 3. 192D Moment Resisting Steel Frame

Section properties of 2D moment resistant frame are as mentioned in following fig3.19and table.3.3

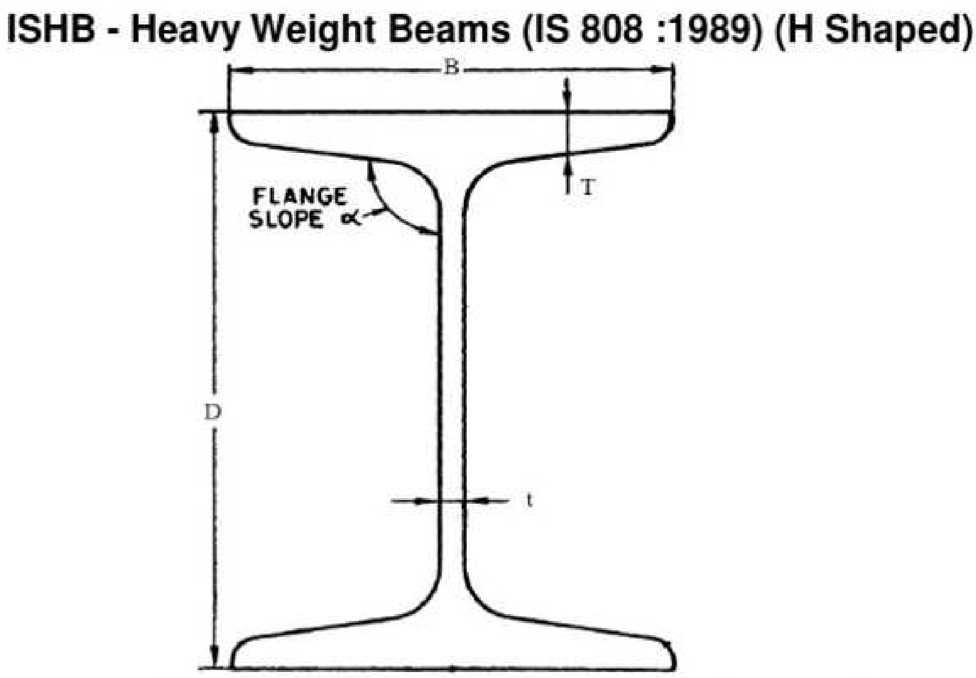


Fig 3. 20I-section

Table 3. 3Steel Section Properties Details

Floor	Beam	Column
Ground floor	ISHB225	ISHB450
First floor	ISHB225	ISHB450
Second floor	ISHB225	ISHB450
Third Floor	ISHB225	ISHB450

Table 3. 4Steel Section Geometric Properties

SIZE	Kg/m	Height D(mm)	Width B(mm)	Web Thickness t (mm)	Flange Thickness T (mm)	Flange Slope α°
ISHB225	43.10	225	225	6.50	9.10	94.00
ISHB450	87.20	450	250	9.80	13.70	94.00

All other inputs are given to perform pushover analysis in STAAD Pro are as mentioned in 3.1.3 of the thesis.

Model: 3> 3D Concentric Braced Frames

a. **Steel Frame Without Bracing**

3D steel frame model of G+ 3 storeys without bracings is considered for performance based seismic design analysis. 3D model of steel frame without bracing developed in STAAD Pro as shown in below fig. 3.21

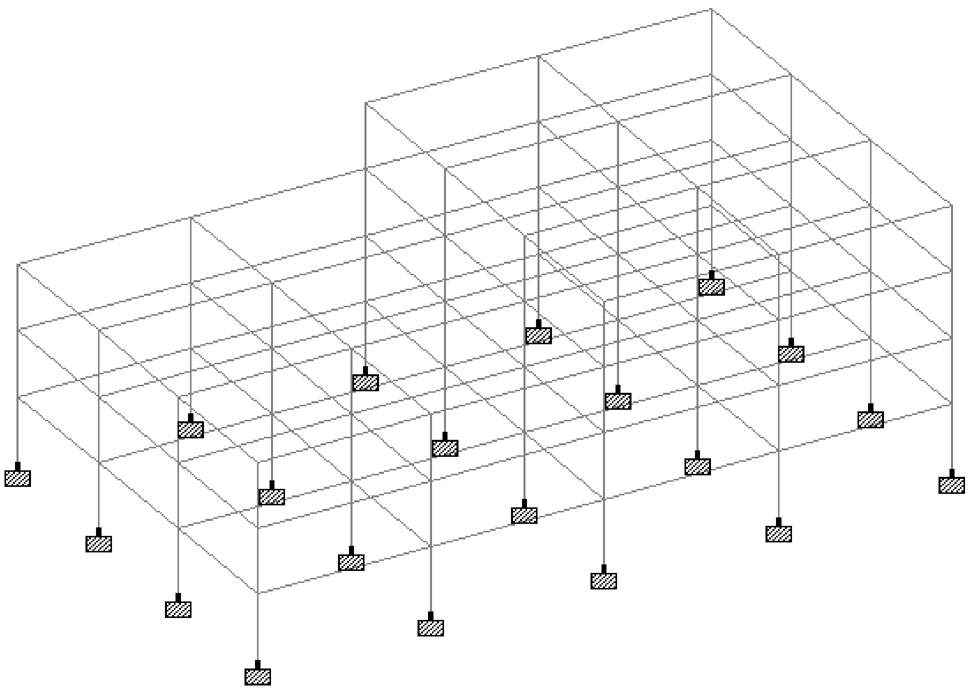


Fig 3. 21Steel Frame Without Bracing

Geometry of steel frame without bracing model in STAAD Pro is as mentioned in following table. 3.5

Table 3. 5Geometry of Steel Frame Without Bracing

NODE	X (m)	Y (m)	Z (m)
1	0	0	0
2	0	0	20
3	0	0	40
4	0	0	60
5	25	0	0
6	25	0	20
7	25	0	40
8	25	0	60
9	50	0	0
10	50	0	20
11	50	0	40
12	50	0	60
13	75	0	0
14	75	0	20
15	75	0	40
16	75	0	60
17	100	0	0
18	100	0	20
19	100	0	40
20	100	0	60
21	0	10	0
22	0	10	20
23	0	10	40
24	0	10	60
25	25	10	0
26	25	10	20
27	25	10	40
28	25	10	60
29	50	10	0
30	50	10	20
31	50	10	40
32	50	10	60
33	75	10	0
34	75	10	20
35	75	10	40
36	75	10	60
37	100	10	0
38	100	10	20
39	100	10	40
40	100	10	60

50	0	20	0
51	0	20	20
52	0	20	40
53	0	20	60
62	25	20	0
63	25	20	20
64	25	20	40
65	25	20	60
74	50	20	0
75	50	20	20
76	50	20	40
77	50	20	60
86	75	20	0
87	75	20	20
88	75	20	40
89	75	20	60
98	100	20	0
99	100	20	20
100	100	20	40
101	100	20	60
104	0	30	0
105	0	30	20
106	0	30	40
107	0	30	60
116	25	30	0
117	25	30	20
118	25	30	40
119	25	30	60
128	50	30	0
129	50	30	20
130	50	30	40
131	50	30	60
140	75	30	0
141	75	30	20
142	75	30	40
143	75	30	60
152	100	30	0
153	100	30	20
154	100	30	40
155	100	30	60
156	50	40	0
157	50	40	20
158	50	40	40
159	50	40	60

168	75	40	0
169	75	40	20
170	75	40	40
171	75	40	60
180	100	40	0
181	100	40	20
182	100	40	40
183	100	40	60

Material used to build 3D steel frame without bracing model issteel as shown in below fig. 3.22

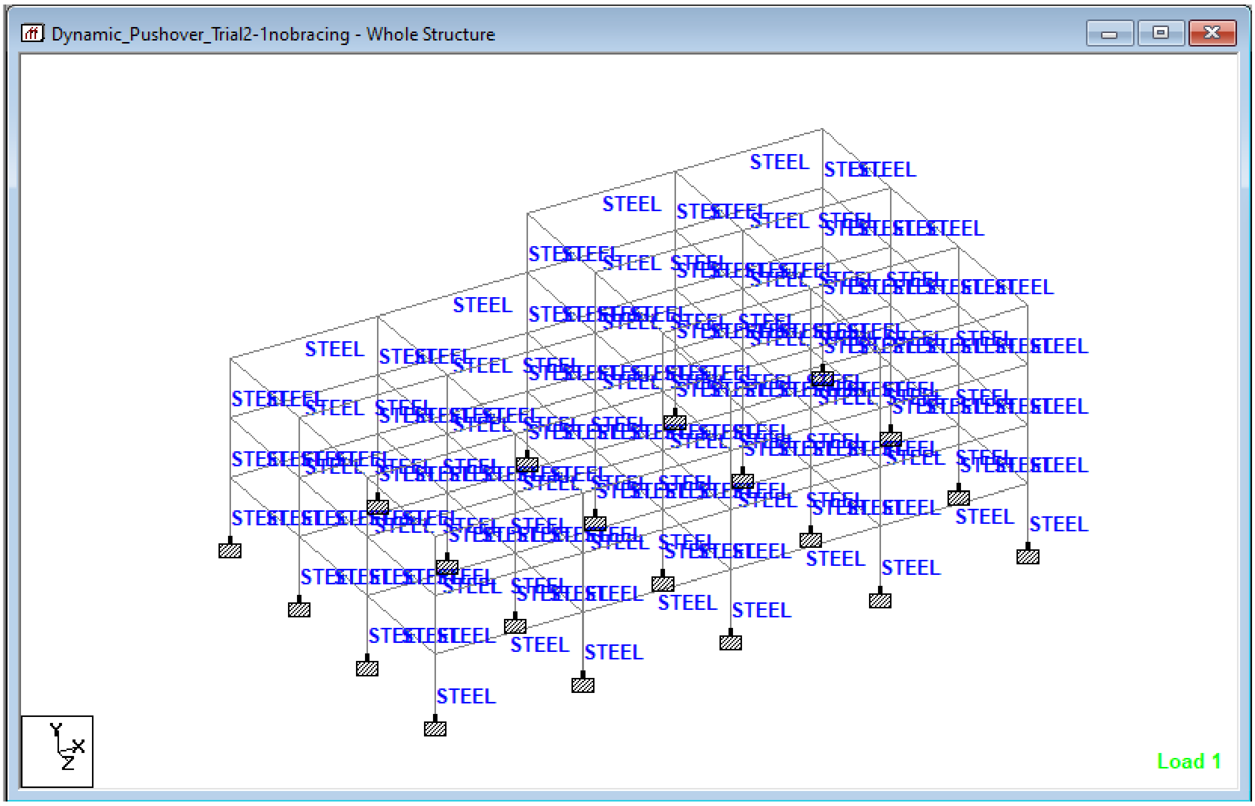


Fig 3. 22Material Properties of Steel Frame Without Bracings

Section properties of Beams and columns in 3D steel frame without bracings are as shown in below fig 3.23

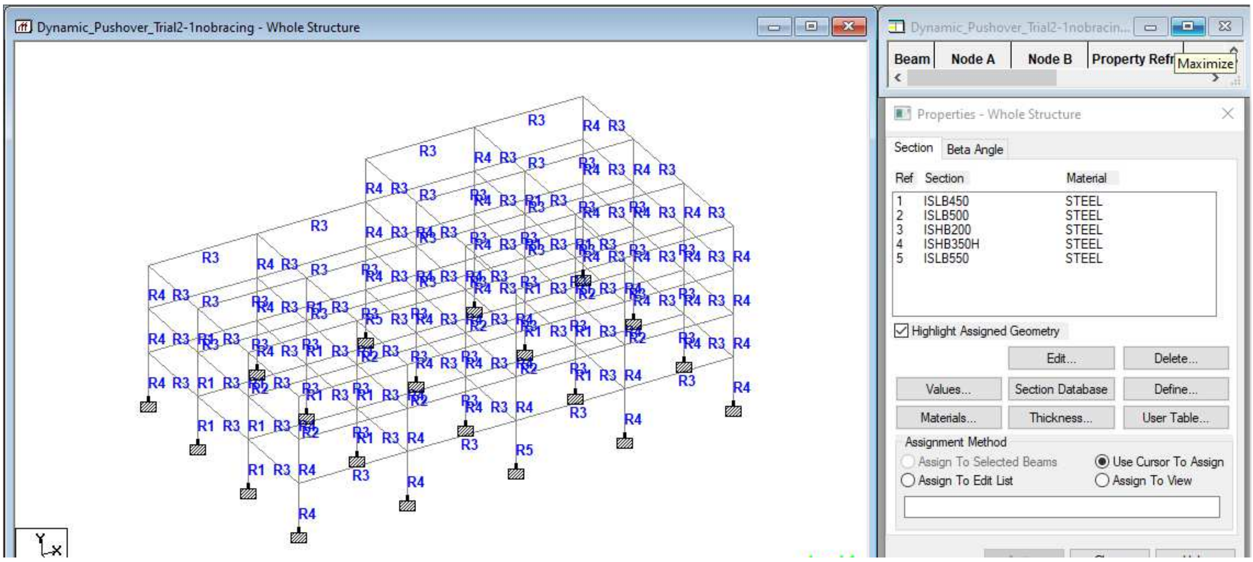


Fig 3. 23Section Property Window

Load case details to perform pushover analysis in STAAD Pro Advanced are followed as mentioned in 3.1.3 of thesis.

After analysis results are recorded in tabular and graphical format.

b. Steel Frame with External Bracing

3D steel frame model of G+ 4 storeys with external bracings philosophy is considered for performance based seismic design analysis. 3D model of steel frame with external bracings developed in STAAD Pro as shown in below fig.3.24

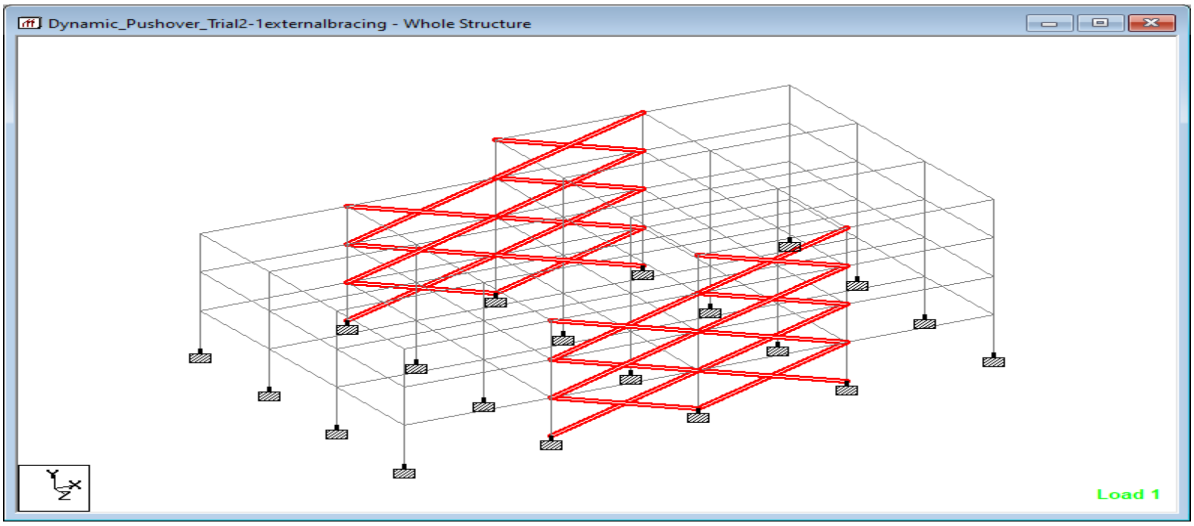


Fig 3. 24External Bracing in 3D Steel Frame

Material used to build 3D steel frame with external bracings model is steel as shown in below fig.3.25

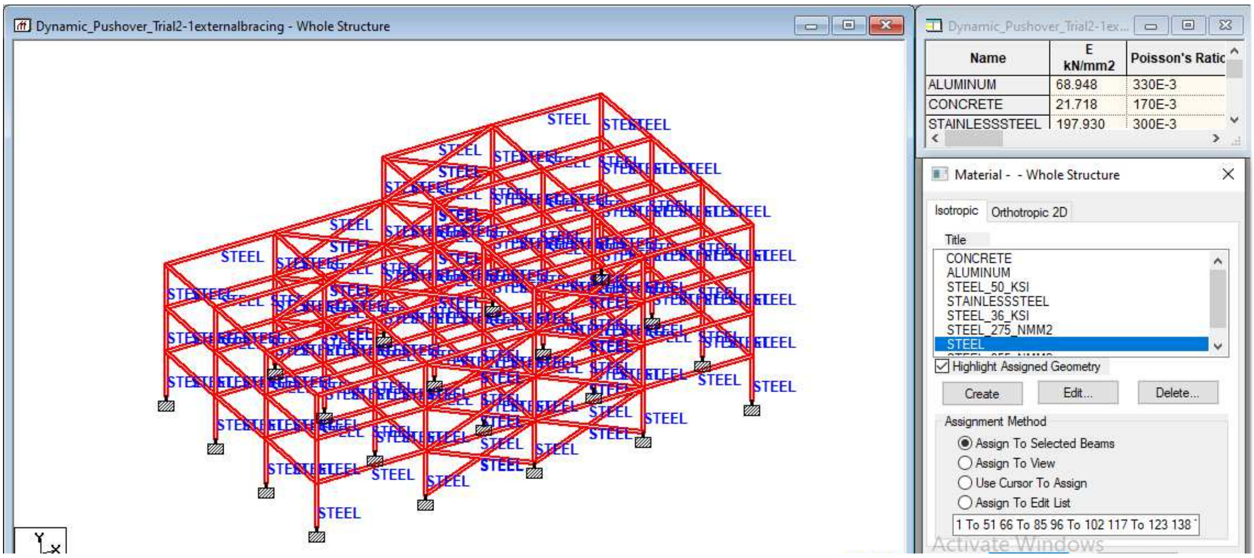


Fig 3. 25Material Property Details of 3D Externally Braced Steed Frame

Section properties of Beams and columns in 3D steel frame with external bracings frame are as shown in below fig.3.26

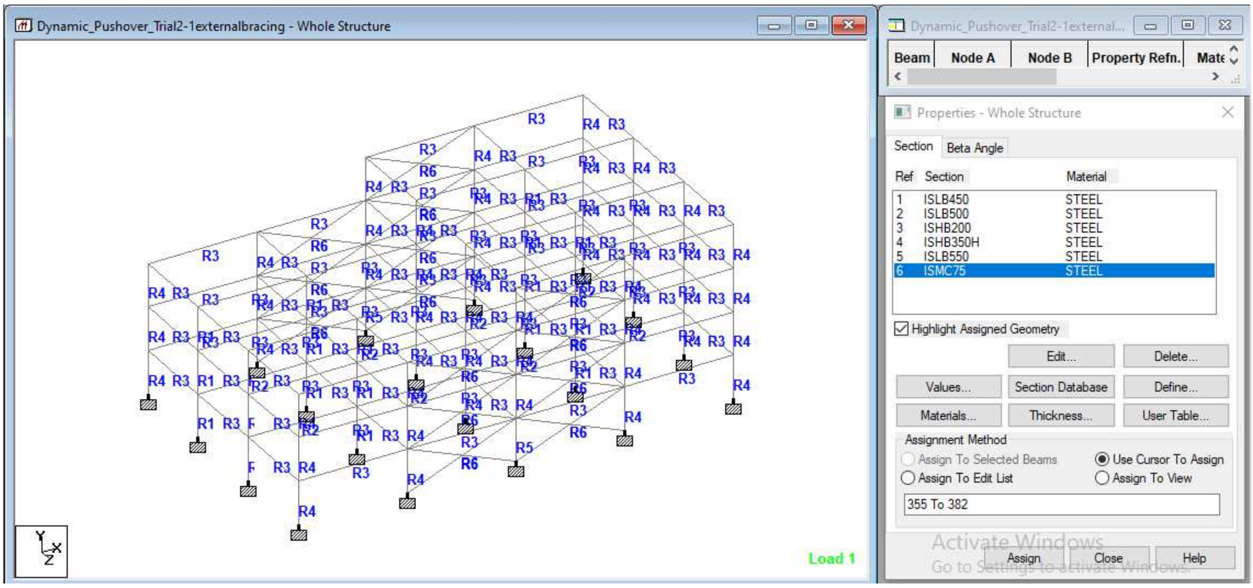


Fig 3. 26Section Properties of 3D Externally Braced Steel Frame

Load case details to perform pushover analysis in STAAD Pro Advanced are followed as mentioned in 3.1.3 of thesis.

After analysis results are recorded in tabular and graphical format.

c. Steel Frame with Optimum Bracing

3D steel frame model of G+ 4 storeys with optimum bracing philosophy is considered for performance based seismic design analysis. 3D model of steel frame with optimum bracings developed in STAAD Pro as shown in below fig.3.27

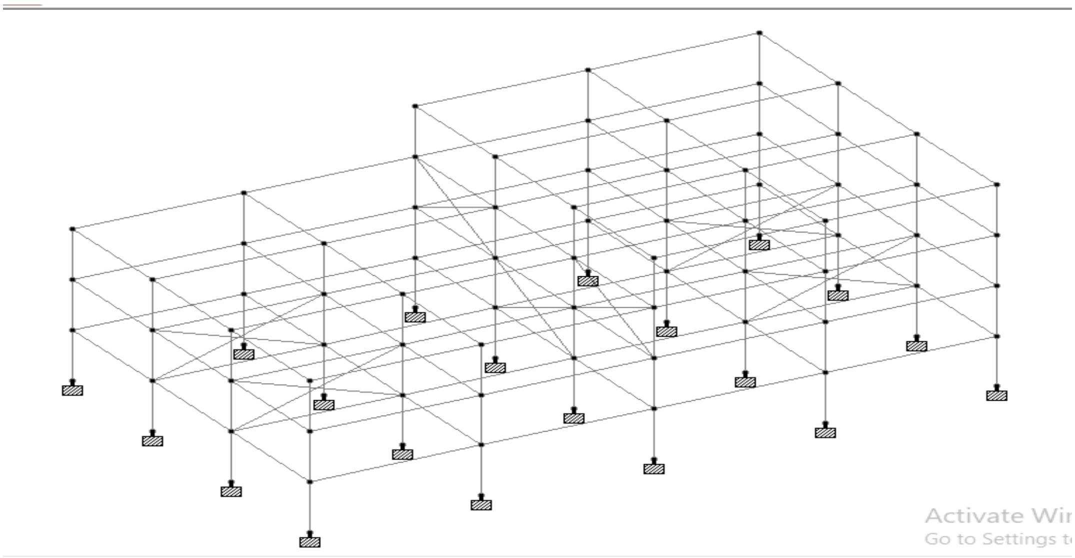


Fig 3. 27Steel Frame with Optimum Bracing

Geometry of steel frame with optimum bracing model in STAAD Pro is as mentioned in following table 3.6

Table 3. 6Geometry of Steel Frame Without Bracing

NODE	X (m)	Y (m)	Z (m)
1	0	0	0
2	0	0	20
3	0	0	40
4	0	0	60
5	25	0	0
6	25	0	20
7	25	0	40
8	25	0	60
9	50	0	0
10	50	0	20
11	50	0	40
12	50	0	60
13	75	0	0
14	75	0	20
15	75	0	40
16	75	0	60
17	100	0	0
18	100	0	20
19	100	0	40

20	100	0	60
21	0	10	0
22	0	10	20
23	0	10	40
24	0	10	60
25	25	10	0
26	25	10	20
27	25	10	40
28	25	10	60
29	50	10	0
30	50	10	20
31	50	10	40
32	50	10	60
33	75	10	0
34	75	10	20
35	75	10	40
36	75	10	60
37	100	10	0
38	100	10	20
39	100	10	40
40	100	10	60
50	0	20	0
51	0	20	20
52	0	20	40
53	0	20	60
62	25	20	0
63	25	20	20
64	25	20	40
65	25	20	60
74	50	20	0
75	50	20	20
76	50	20	40
77	50	20	60
86	75	20	0
87	75	20	20
88	75	20	40
89	75	20	60
98	100	20	0
99	100	20	20
100	100	20	40
101	100	20	60
104	0	30	0
105	0	30	20
106	0	30	40

107	0	30	60
116	25	30	0
117	25	30	20
118	25	30	40
119	25	30	60
128	50	30	0
129	50	30	20
130	50	30	40
131	50	30	60
140	75	30	0
141	75	30	20
142	75	30	40
143	75	30	60
152	100	30	0
153	100	30	20
154	100	30	40
155	100	30	60
156	50	40	0
157	50	40	20
158	50	40	40
159	50	40	60
168	75	40	0
169	75	40	20
170	75	40	40
171	75	40	60
180	100	40	0
181	100	40	20
182	100	40	40
183	100	40	60

Material used to build 3D steel frame with optimum bracing model is steel as shown in below fig.3.28

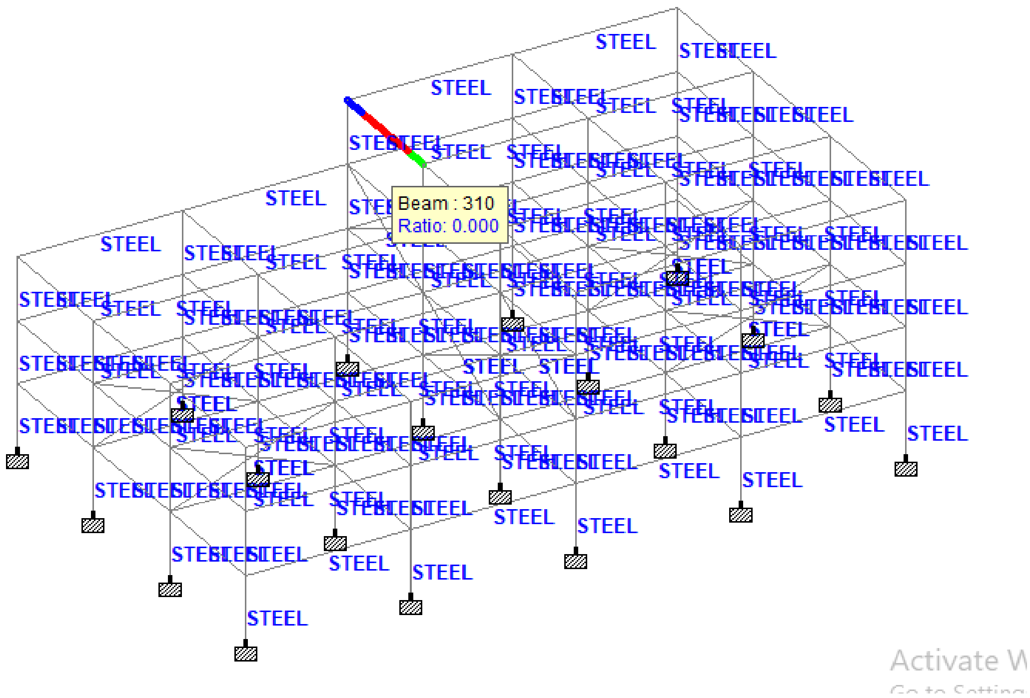


Fig 3. 28Material Property of Frame with Optimum Bracing

Section properties of Beams and columns in 3D steel frame with optimum bracing are as shown in below fig.3.29

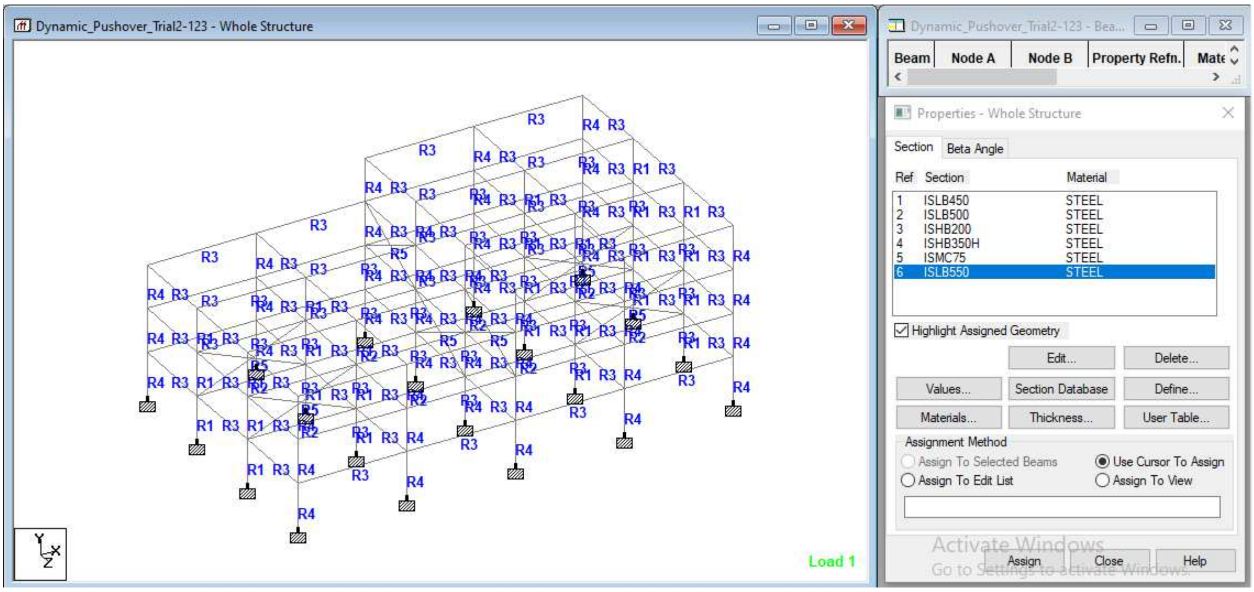


Fig 3. 29Section Properties of Steel Frame with Optimum Bracings

Load case details to perform pushover analysis in STAAD Pro Advanced are followed as mentioned in 3.1.3 of thesis.

After analysis results are recorded in tabular and graphical format

CHAPTER 4

RESULTS & DISCUSSION

4.1 2D Moment Resisting Frame

Total base shear of 314 KN is applied in step by step pushover analysis. Total steps to apply 314 KN base shears are 51 which caused maximum displacement of 130 mm in 2D steel moment resisting frame. Following is capacity curve of 2D moment resisting frame. On X- axis displacement at controlled joint is shown while on Y- axis base shear is represented.

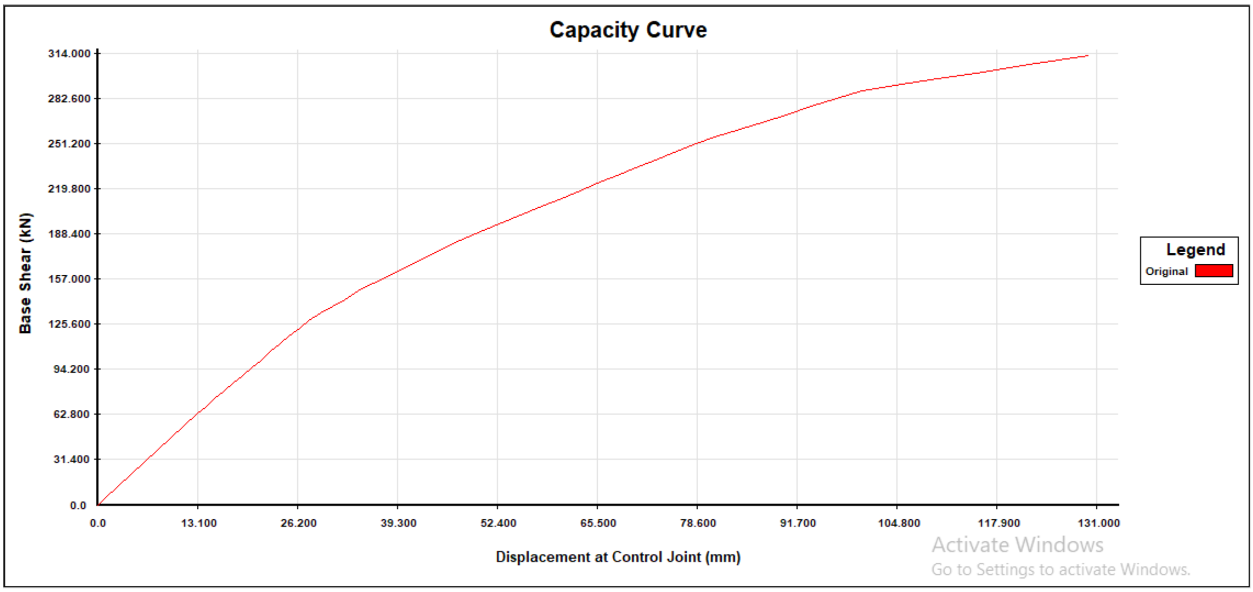


Fig 4. 1Capacity Curve of 2D moment resisting frame

Fig.1

Base shear is applied step by step in total 51 steps. During each step displacement caused at control joint is recoded which gives idea about the effect of push loads on steel frame. Following table contains Displacement results of 2D steel frame during each load step.

Table 4. 1Displacement due to Base Shear in 2D Moment Resisting Frame

Load Step (%s)	Displacement mm	Base Shear kip
1	0	0
2	1.112	1.236
3	2.71	3.013
4	13.054	14.358
5	14.165	15.48
6	15.39	16.716
7	16.614	17.951

8	17.838	19.187
9	19.062	20.423
10	20.287	21.659
11	21.511	22.894
12	22.735	24.13
13	23.959	25.366
14	25.184	26.601
15	26.535	27.837
16	27.768	28.956
17	29.278	30.183
18	30.859	31.289
19	32.409	32.361
20	34.176	33.583
21	36.211	34.819
22	38.018	35.904
23	40.053	37.126
24	42.111	38.362
25	44.17	39.597
26	46.228	40.833
27	48.533	42.069
28	50.614	43.173
29	53.105	44.395
30	55.45	45.538
31	57.968	46.767
32	60.501	48.002
33	63.035	49.238
34	65.569	50.474
35	68.102	51.709
36	70.636	52.945
37	73.17	54.181
38	75.703	55.416
39	78.474	56.652
40	81.03	57.782
41	84.263	59.007

42	87.107	60.066
43	90.368	61.28
44	93.687	62.516
45	97.005	63.752
46	100.323	64.988
47	106.434	66.223
48	110.172	66.894
49	116.282	67.991
50	123.166	69.227
51	130.051	70.462

All relative displacement in beams is as shown in following table 4.2

Table 4. 2Maximum Relative Displacement in Beams of 2D Moment Resisting Frame

Beam	L/C	Length	Dist	Max y	Dist m	Max mm	Dist	Span/Max
		m	m	mm			m	
1	1	3	2.25	0.161	2	0.161	2	>10000
2	1	3	1.25	-2.143	1.5	2.143	1.5	1400
3	1	3	1.75	0	0	0	1.75	
4	1	3	2.5	-0.087	0.75	0.087	0.75	>10000
5	1	3	2.25	-2.126	1.5	2.126	1.5	1411
6	1	3	1.5	0	0	0	1.5	
7	1	3	1.75	-0.087	0.75	0.087	0.75	>10000
8	1	3	1.5	-2.105	1.5	2.105	1.5	1425
9	1	3	1.25	0	0	0	1.25	
10	1	3	1.25	0.286	2	0.286	2	>10000
11	1	3	2.75	-2.321	1.5	2.321	1.5	1292
12	1	3	2	0	0	0.001	2	
13	1	3	2.5	-2.142	1.5	2.142	1.5	1400

14	1	3	2.75	-2.127	1.5	2.127	1.5	1411
15	1	3	2.5	-2.102	1.5	2.102	1.5	1427
16	1	3	2.5	-2.329	1.5	2.329	1.5	1288
17	1	3	2.25	0.161	1	0.161	1	>10000
18	1	3	1.75	-0.087	2.25	0.087	2.25	>10000
19	1	3	2.75	-0.088	2.25	0.088	2.25	>10000
20	1	3	1	0.286	1	0.286	1	>10000

Fig:4.2show the displacement in 2D moment resisting frame after the application of pushover loads.

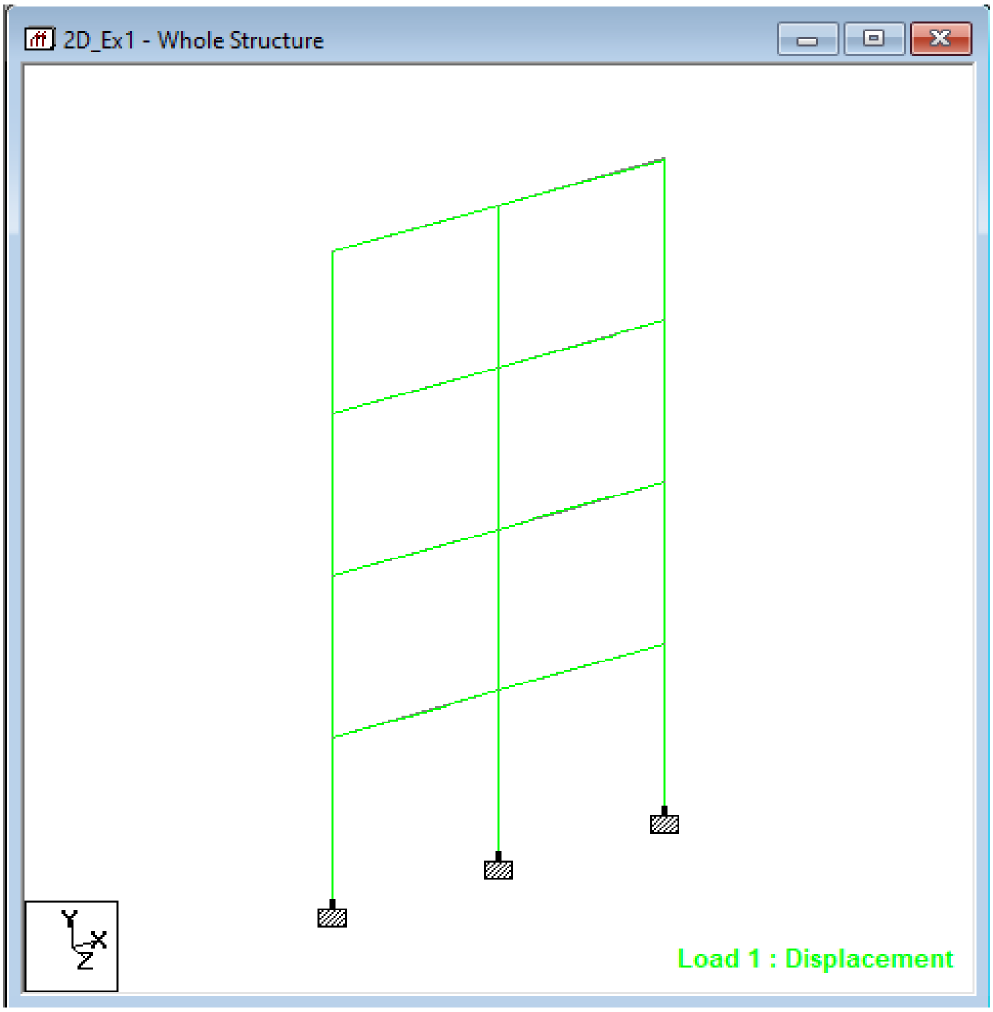


Fig 4. 2Displacement in 2D Moment Resisting Frame

Table4.2 contains data of maximum bending moment caused in 2D moment resisting frame due to considered earthquake loads. Fig shows the Bending moment diagram (BMD) of 2D moment resisting frame.

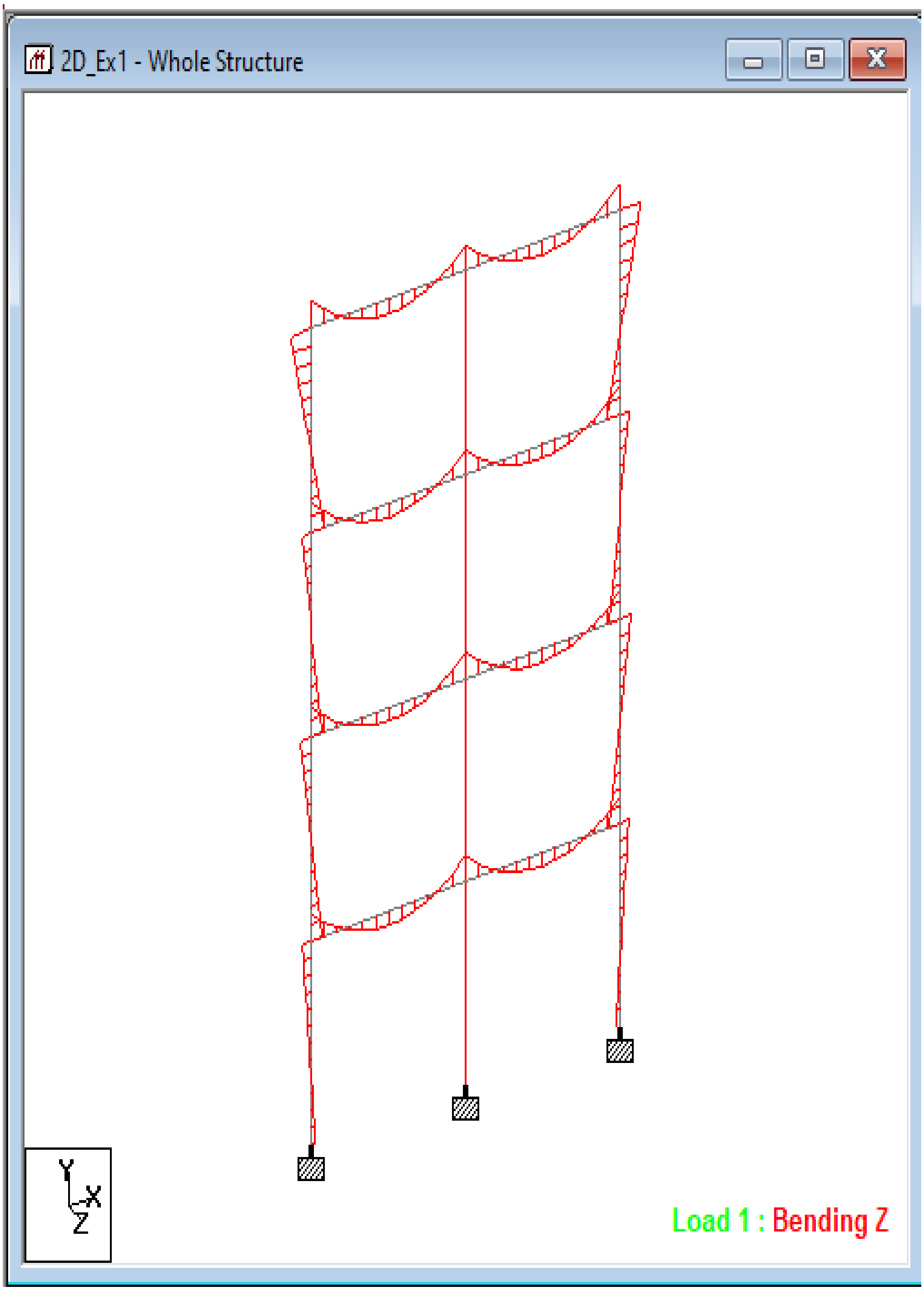


Fig 4. 3Bending Moment Diagram of 2D Moment Resisting Frame

Details of performance of 2D moment resisting frame after application of pushover loads discussed here. As the base shear applied displacement is recorded by the software. Also to find out in which performance level steel frame is at the particular base shear hinge formation status is also recorded by STAAD Pro. Advanced. At load step: 1 total bases shear of 0.000001KN is applied. At that point very less displacement and no hinge formation observed at controlled joint. Following fig4.4 shows the displacement and hinge formation status when Base shear is 0.000001KN.

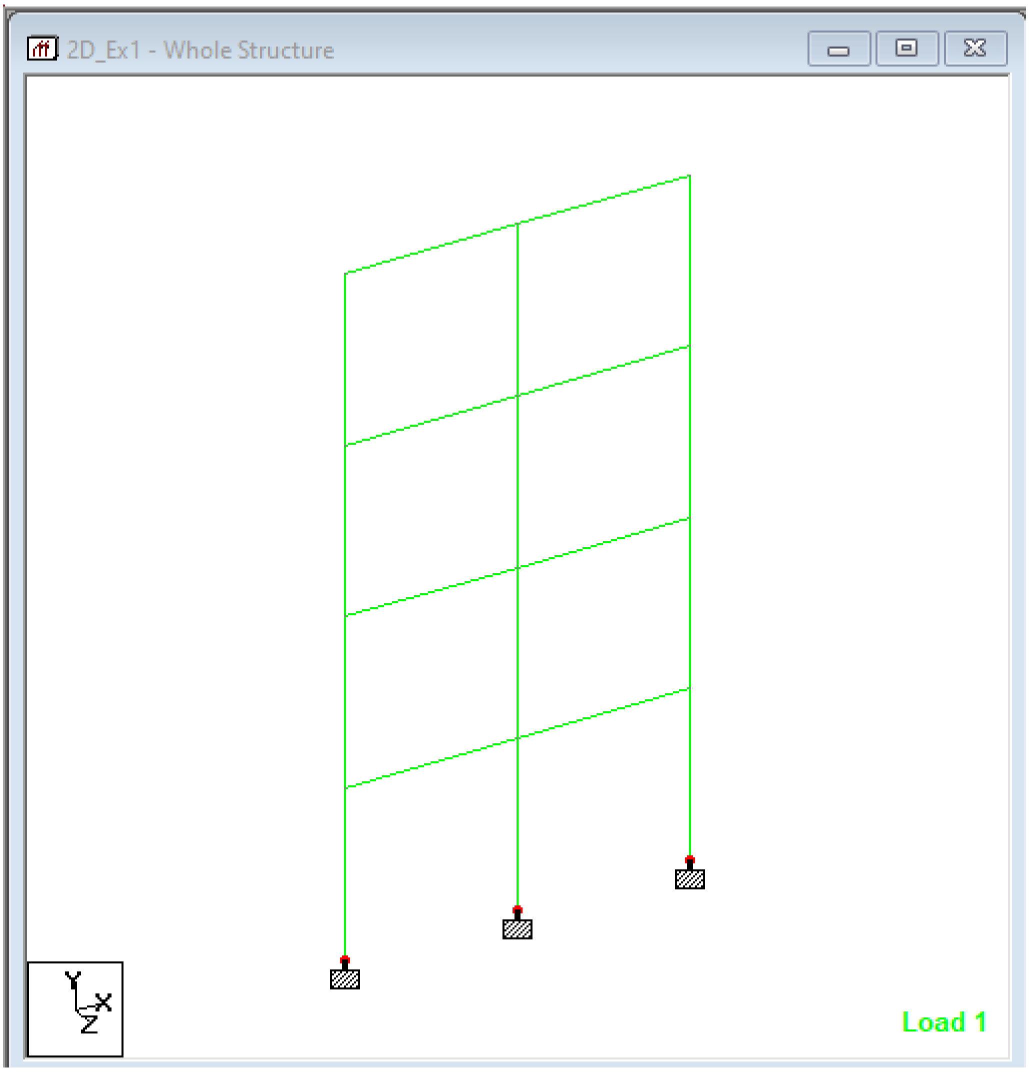


Fig 4. 4Hinge Formation Status when Base Shear is 0.000001KN

As no plastic hinge developed in the frame, frame lies in safe performance level. Hence no other performance level is added here. As no hinge formation is observed in this load condition hence every beam is performing linearly. Following table 10 shows the Hinge Status of 2D moment resisting frame.

Table 4. 3Hinge Status of 2D Moment Resisting Frame

			Hinge Location Status					
Beam	**Status**	**Dir (Local)**	**Section m**	**Status**	**Section m**	**Status**	**Section m**	**Status**
1-20	Linear	-	-	-	-	-	-	-

During the lateral load step: 4, base shear of 63.867651KN is applied on 2D moment resisting frame. At this stage beam no: 3 failed as shown in fig:4.5

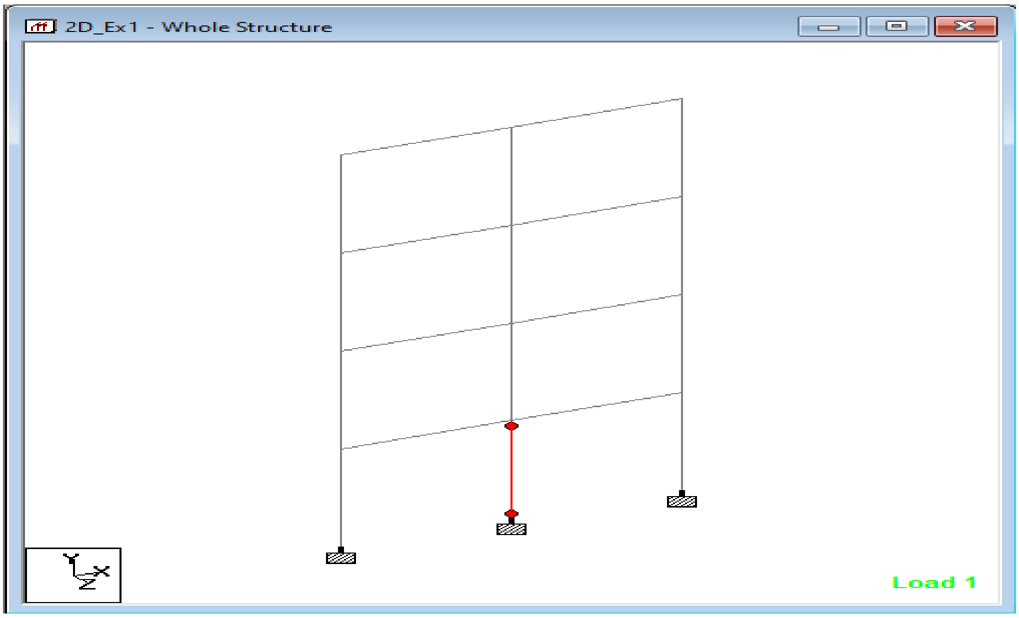


Fig 4. 5Hinge status when Base Shear is 63.867651KN

Even though beam no 3 collapsed frame is further analyzed as frame can till sustain on remaining two end columns. Displacement recorded at that point is as shown in Fig:4.6

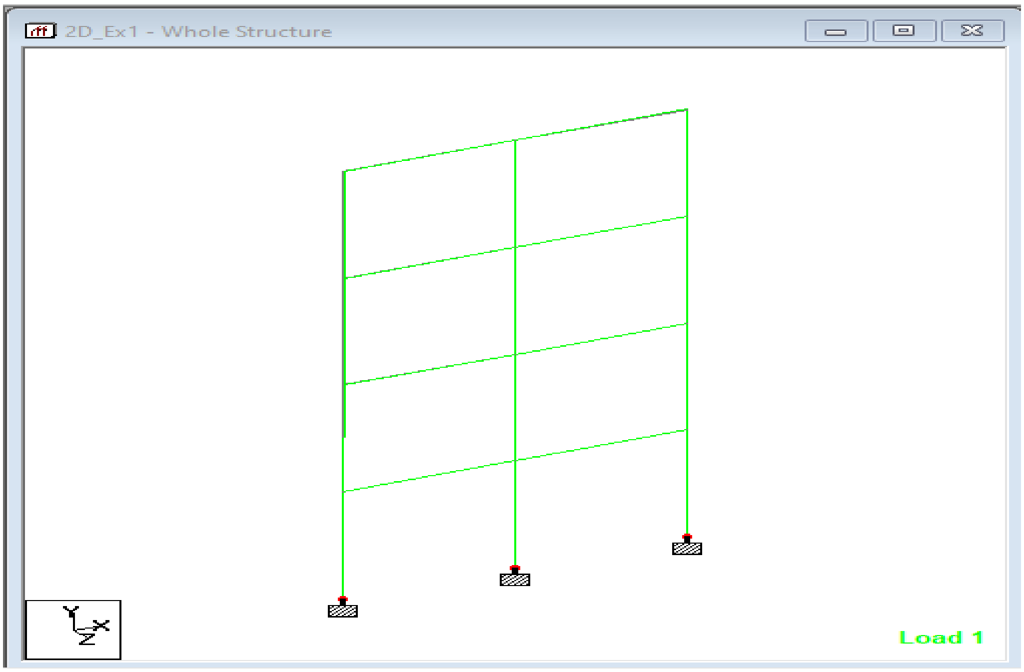


Fig 4. 6Displacement when Base Shear is 63.867651KN

At load step: 5 when base shear is 123.528230KN plastic hinge developed in beam no 14. At this stage beam no 14 started performing nonlinearly as shown in Fig: 4.7

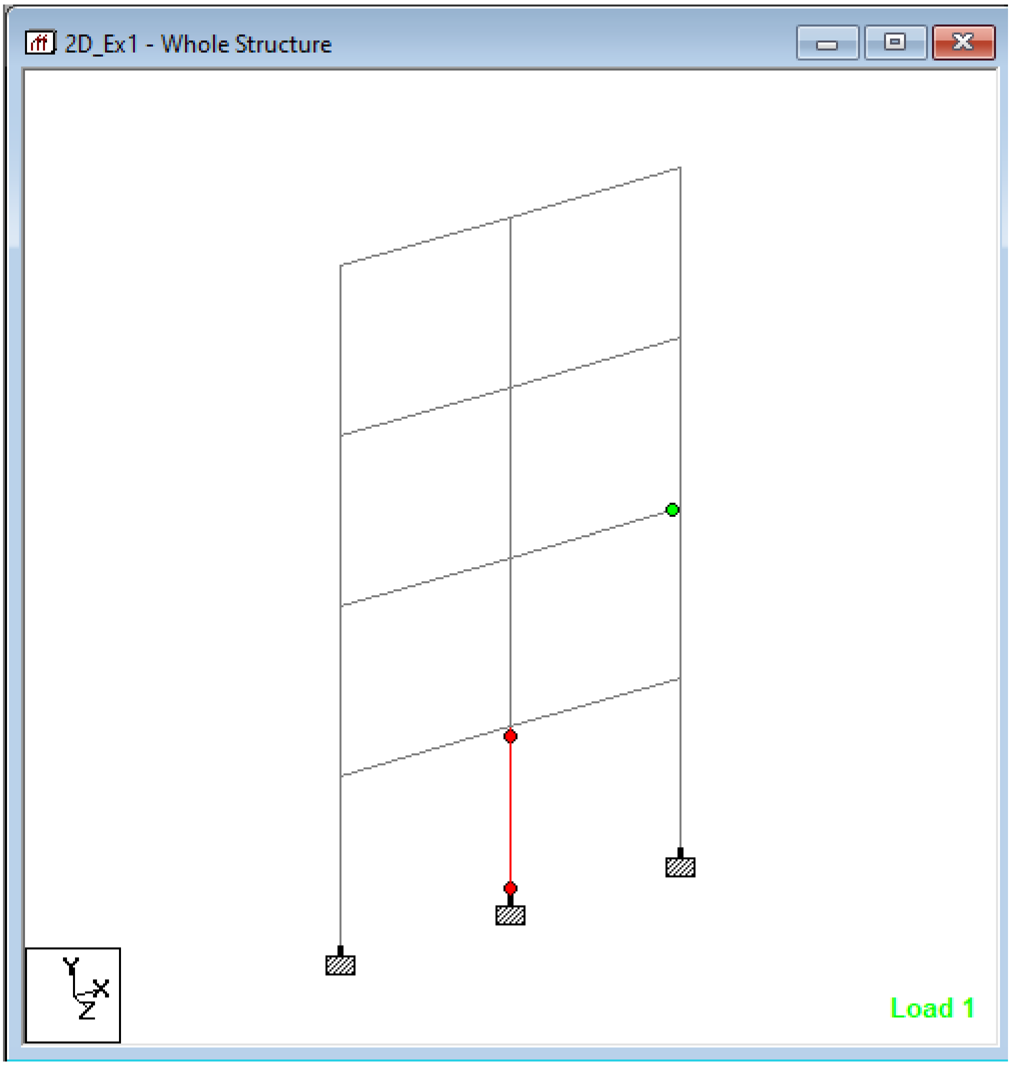


Fig 4. 7Hinge Status when Base Shear is 123.528230KN

Plastic hinge status of each beam in 2D moment resisting frame at load step 15, base shear 123.528230KN is as shown in Table 4.4

Table 4. 4Hinge Location Status of 2D Moment Frame

Beam	Status	Dir (Local)	Section m	Status
1-2	Linear			
3	Inactive			
4-13	Linear			
14	Nonlinear	Z	3	<= IO
15-20	Linear			

At the same stage displacement recorded at controlled joint is 26.536mm as shown in Fig:4.8

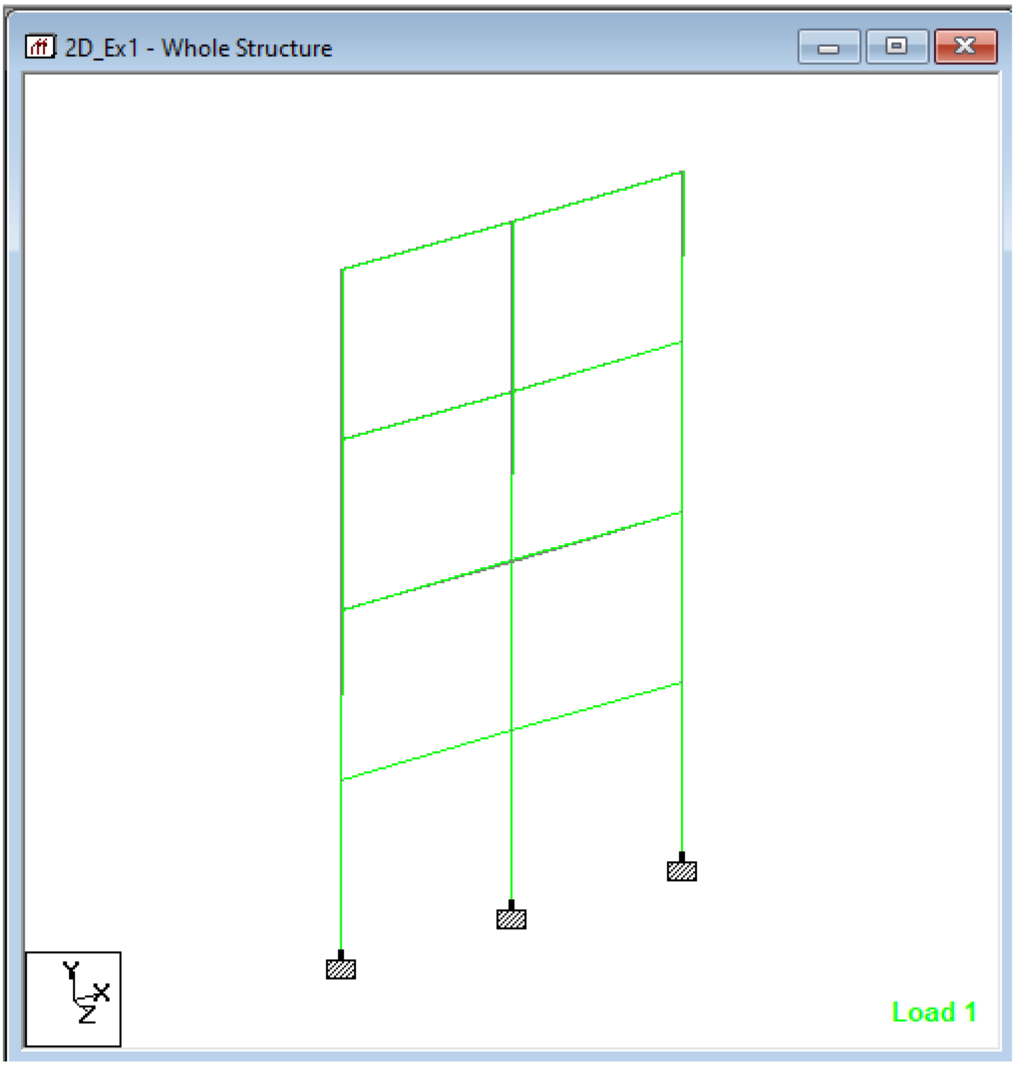


Fig 4. 8Displacement when Base Shear is 123.528230KN

Plastic hinge formation continued in 2D moment resisting frame until base shear reached the 313.432228KN value. At the load step 51, Beam no : 1,2,5,8,11,13,14,15,16 and 17 are started performing in IO performance level. At this point maximum beam started acting nonlinearly. Plastic hinge formation diagram is as shown in Fig:46 and Hinge location status in beam is computed in Table 4.5

Table 4. 5Hinge Location Status when Base Shear is 313.432228KN

Beam	Status	Dir (Local)	Section m	Status	Section m	Status
1	Nonlinear	Z	0	<= IO		
2	Nonlinear	Z			3	<= IO

3	Inactive					
4	Linear					
5	Nonlinear	Z			3	<= IO
6-7	Linear					
8	Nonlinear	Z			3	<= IO
9-10	Linear					
11	Nonlinear	Z			3	<= IO
12	Linear					
13	Nonlinear	Z			3	<= IO
14	Nonlinear	Z			3	<= IO
15	Nonlinear	Z			3	<= IO
16	Nonlinear	Z			3	<= IO
17	Nonlinear	Z			3	<= IO
18-20	Linear					

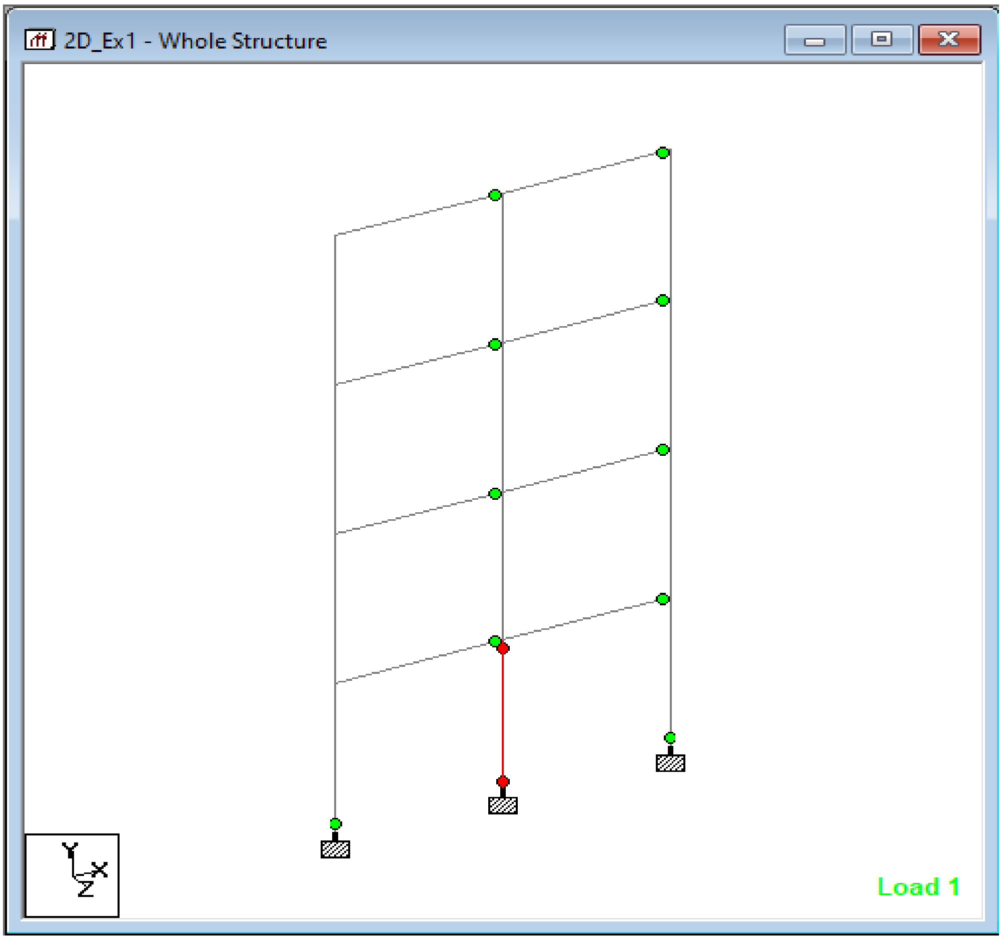


Fig 4. 9Hinge Status when Base Shear is 313.432228KN

4.2 3D Concentric Braced Frames

a. Steel Frame Without Bracing

After performing pushover analysis on the G+3 storey steel frame, frame performed linearly up to the base shear of 135.09KN then after it started performing nonlinearly as base shear increased. First plastic hinge shown by green is developed in column 9 and column 12 as shown in figure 47. Performance level at that point is IO. Further push load steps carried out, when base shear is increase up to 3758.82 KN blue colored plastic hinges developed at a column 5, 8, 13and 16 as shown in fig 48 which shows the structure lies in between IO – LS performance level. When base shear reached the value of 4218KN pink colored plastic hinges started developing into the column 5, 8, 13 and 16 as shown in fig 49 which implies that the those structural components lies in LS-CP performance level. At the base shear 4392.60 KN red colored plastic hinges started developing into the column 5, 8, 13 and 16 as shown in fig 8 which implies that the those structural components are in CP level. As the member of structure comes into the CP performance level base shear started redistributing to check the performance of other structural elements. Till that point no member were collapsed. First member column 5 and 8 failed at the base shear 4260.87KN and they are indicted by red color. But entire structure was not failed at those points as the maximum columns lies into the IO performance level as shown in fig50. After distributing and redistributing base shear up to the push load step 173 maximum number of beams and columns are into the CP level and LS level while some of them are collapsed as shown in fig when the redistributed base shear was 2380.60 KN. After that point entire structure will fail as maximum number of columns from base storey was failed as shown in fig 51. Capacity curve for moment frame is as shown in fig 10 in which X-axis indicates the displacement at roof due to base shear indicated on Y- axis.

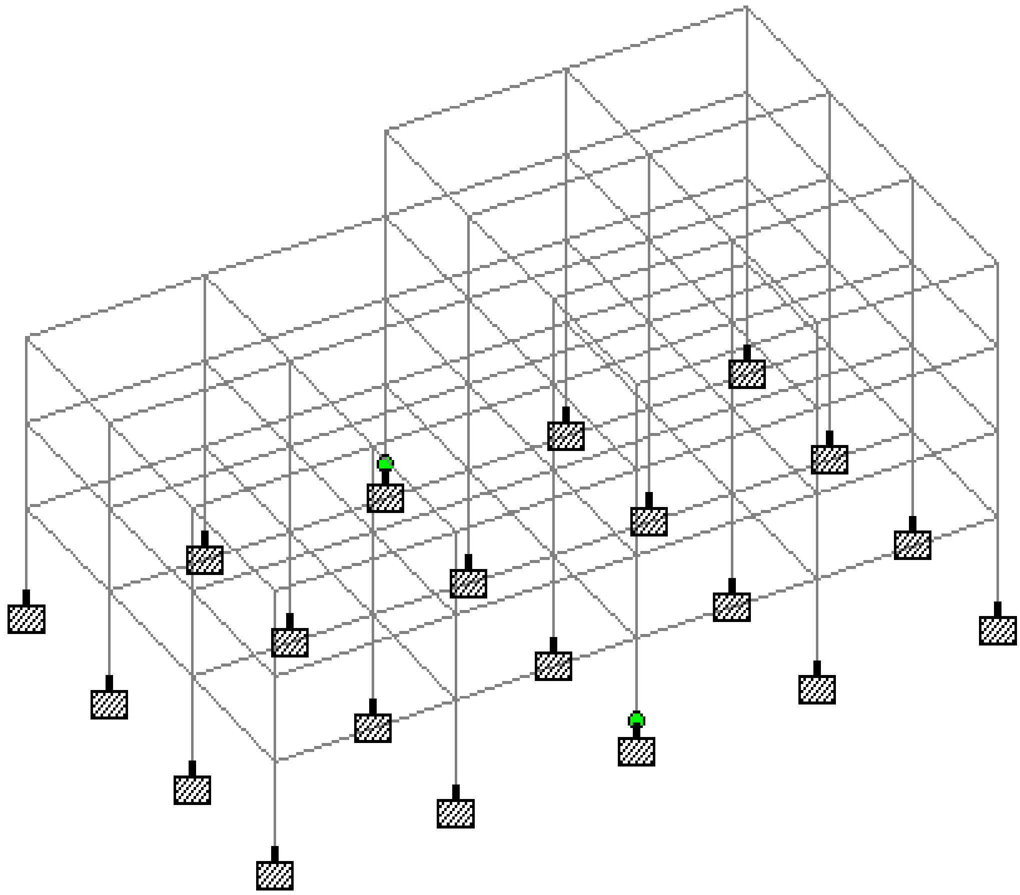


Fig 4. 10Members of Moment Steel Frame in IO Performance Level

Plastic hinge formation at load step: 4 when base shear is 135.097 KN is as computed in Table:4.6

Table 4. 6Hinge Location Status at 135 kn Base Shear

Beam	Status	Direction (Local)	Section m	Status	Section m	Status	Section m	Status
1-8	Linear							
9	Nonlinear	Z					3.048	<= IO
10-11	Linear							
12	Nonlinear	Z					3.048	<= IO
13-354	Linear							

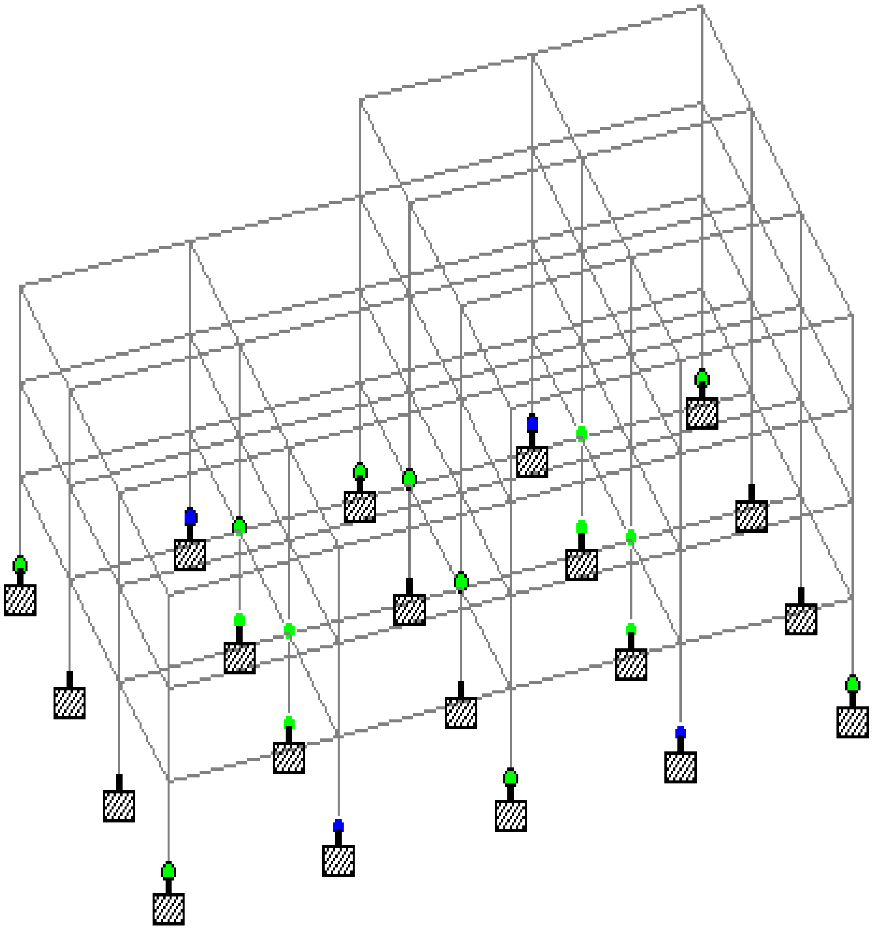


Fig 4. 11Members of Moment Steel Frame in IO-LS Performance Level

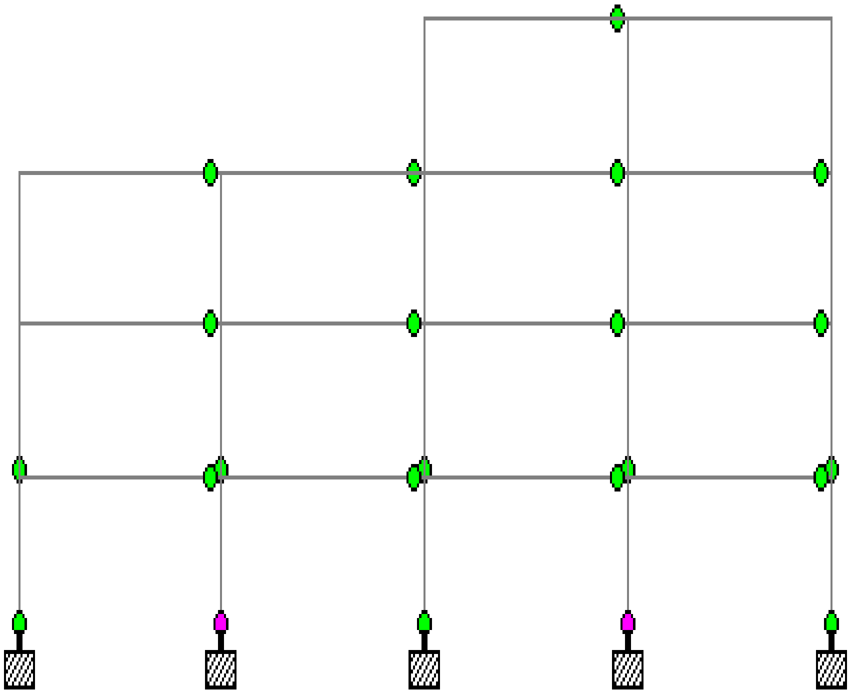


Fig 4. 12Members of Moment Steel Frame in LS-CP

Table:4.7 shows the plastic hinge location status of frame at the pushover load step: 25 when the base shear applied is 3758.822KN and displacement occurred at the controlled joint is 205.432 mm.

Table 4. 7Hinge location Status when base shear is 3758.822KN

Beam	Status	Dir (Local)	Section m	Status
1	Nonlinear	Z	3.048	<= IO
2-3	Linear			
4	Nonlinear	Z	3.048	<= IO
5	Nonlinear	Z	3.048	IO - LS
6	Nonlinear	Z	3.048	<= IO
7	Nonlinear	Z	3.048	<= IO
8	Nonlinear	Z	3.048	IO - LS
9	Nonlinear	Z	3.048	<= IO
10-11	Linear			
12	Nonlinear	Z	3.048	<= IO
13	Nonlinear	Z	3.048	IO - LS
14	Nonlinear	Z	3.048	<= IO
15	Nonlinear	Z	3.048	<= IO
16	Nonlinear	Z	3.048	IO - LS
17	Nonlinear	Z	3.048	<= IO
18-19	Linear			
20	Nonlinear	Z	3.048	<= IO
21-70	Linear			
71	Nonlinear	Z	3.048	<= IO
72	Nonlinear	Z	3.048	<= IO

73-74	Linear			
75	Nonlinear	Z	3.048	<= IO
76	Nonlinear	Z	3.048	<= IO
77-78	Linear			
79	Nonlinear	Z	3.048	<= IO
80	Nonlinear	Z	3.048	<= IO
81-354	Linear			

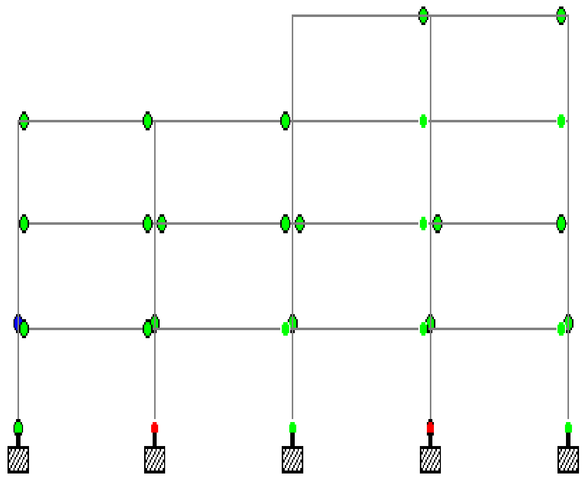


Fig 4. 13Members of Moment Steel Frame in CP

Table 4.8shows the plastic hinge formation status at load step:50 when base shear is 4218.078KN.

Table 4. 8Hinge Location Status when Base Shear is 4218.078KN

Beam	Status	Dir (Local)	Section m	Status	Section m	Status
1	Nonlinear	Z			3.048	<= IO
2	Nonlinear	Z			3.048	<= IO
3	Nonlinear	Z			3.048	<= IO
4	Nonlinear	Z			3.048	<= IO

5	Nonlinear	Z			3.048	LS - CP
6	Nonlinear	Z			3.048	<= IO
7	Nonlinear	Z			3.048	<= IO
8	Nonlinear	Z			3.048	LS - CP
9	Nonlinear	Z			3.048	<= IO
10	Nonlinear	Z			3.048	IO - LS
11	Nonlinear	Z			3.048	IO - LS
12	Nonlinear	Z			3.048	<= IO
13	Nonlinear	Z			3.048	LS - CP
14	Nonlinear	Z			3.048	<= IO
15	Nonlinear	Z			3.048	<= IO
16	Nonlinear	Z			3.048	LS - CP
17	Nonlinear	Z			3.048	<= IO
18	Nonlinear	Z			3.048	<= IO
19	Nonlinear	Z			3.048	<= IO
20	Nonlinear	Z			3.048	<= IO
21-23	Linear					
24	Nonlinear	Z	0	<= IO		
25-26	Linear					
27	Nonlinear	Z			7.62	<= IO
28-30	Linear					
31	Nonlinear	Z			7.62	<= IO
32-33	Linear					
34	Nonlinear	Z			7.62	<= IO

35	Linear					
36-37	Linear					
38	Nonlinear	Z			7.62	<= IO
39-40	Linear					
41	Nonlinear	Z			7.62	<= IO
42-44	Linear					
45	Nonlinear	Z			7.62	<= IO
46-47	Linear					
48	Nonlinear	Z			7.62	<= IO
49-66	Linear					
67	Nonlinear	Z			3.048	<= IO
68	Nonlinear	Z			3.048	<= IO
69-70	Linear					
71	Nonlinear	Z			3.048	<= IO
72	Nonlinear	Z			3.048	<= IO
73-74	Linear					
75	Nonlinear	Z			3.048	<= IO
76	Nonlinear	Z			3.048	<= IO
77-78	Linear					
79	Nonlinear	Z			3.048	<= IO
80	Nonlinear	Z			3.048	<= IO
81-82	Linear					
83	Nonlinear	Z			3.048	<= IO
84	Nonlinear	Z			3.048	<= IO

85-98	Linear					
100	Nonlinear	Z			7.62	<= IO
101	Nonlinear	Z			7.62	<= IO
102	Nonlinear	Z			7.62	<= IO
117-119	Linear					
120	Nonlinear	Z			7.62	<= IO
121	Nonlinear	Z			7.62	<= IO
122	Nonlinear	Z			7.62	<= IO
123	Nonlinear	Z			7.62	<= IO
138-140	Linear					
141	Nonlinear	Z			7.62	<= IO
142	Nonlinear	Z			7.62	<= IO
143	Nonlinear	Z			7.62	<= IO
144	Nonlinear	Z			7.62	<= IO
159-161	Linear					
162	Nonlinear	Z			7.62	<= IO
163	Nonlinear	Z			7.62	<= IO
164	Nonlinear	Z			7.62	<= IO
165	Nonlinear	Z			7.62	<= IO
180-212	Linear					
214	Nonlinear	Z			7.62	<= IO
215	Nonlinear	Z			7.62	<= IO
216	Nonlinear	Z			7.62	<= IO
217	Nonlinear	Z			7.62	<= IO

232-235	Linear					
236	Nonlinear	Z			7.62	<= IO
237	Nonlinear	Z			7.62	<= IO
238-255	Linear					
256	Nonlinear	Z			7.62	<= IO
257	Nonlinear	Z			7.62	<= IO
258	Nonlinear	Z			7.62	<= IO
259	Nonlinear	Z			7.62	<= IO
274-276	Linear					
277	Nonlinear	Z			7.62	<= IO
278	Nonlinear	Z			7.62	<= IO
279	Nonlinear	Z			7.62	<= IO
280	Nonlinear	Z			7.62	<= IO
295-313	Linear					
314	Nonlinear	Z			7.62	<= IO
315	Nonlinear	Z			7.62	<= IO
316-354	Linear					

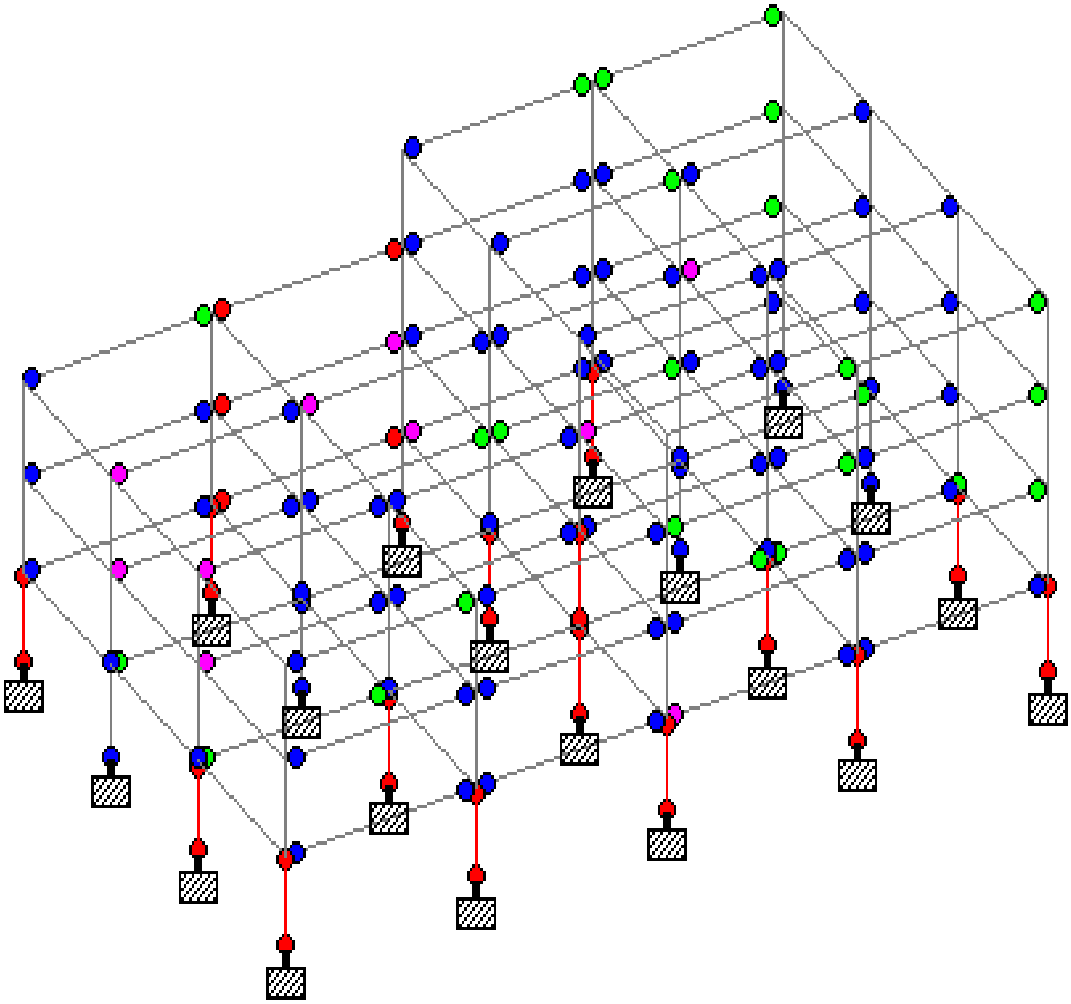


Fig 4. 14Members of Moment Steel Frame Failed in Pushover Analysis

Plastic hinge location status in load step:173 when base shear is 2380.605 KN is computed in Tabel 4.9

Table 4. 9Hinge location Status when Base Shear is 2380.605KN.

Beam	Status	Dir (Local)	Section m	Status	Section m	Status
1	Inactive					
2	Nonlinear	Z			3.048	IO - LS
3-5	Inactive					
6	Nonlinear	Z	0	IO - LS	3.048	IO - LS
7-8	Inactive					

9	Nonlinear	Z			3.048	>= CP
10-13	Inactive					
14	Nonlinear	Z	0	IO - LS	3.048	IO - LS
15-16	Inactive					
17	Nonlinear	Z			3.048	IO - LS
18	Nonlinear	Z			3.048	IO - LS
19-20	Inactive					
21-23	Linear					
24	Nonlinear	Z	0	IO - LS	7.62	IO - LS
25	Nonlinear	Z	0	<= IO		
26	Nonlinear	Z	0	<= IO	7.62	<= IO
27	Nonlinear	Z	0	IO - LS	7.62	IO - LS
28-30	Linear					
31	Nonlinear	Z	0	>= CP	7.62	>= CP
32-33	Linear					
34	Nonlinear	Z	0	IO - LS	7.62	IO - LS
35-37	Linear					
38	Nonlinear	Z	0	LS - CP	7.62	IO - LS
39	Linear					
40	Nonlinear	Z			7.62	<= IO
41	Nonlinear	Z	0	LS - CP	7.62	IO - LS
42-44	Linear					

45	Nonlinear	Z	0	IO - LS	7.62	IO - LS
46	Nonlinear	Z			7.62	<= IO
47	Nonlinear	Z	0	<= IO	7.62	IO - LS
48	Nonlinear	Z	0	IO - LS	7.62	IO - LS
49-66	Linear					
67	Nonlinear	Z			3.048	IO - LS
68	Nonlinear	Z			3.048	IO - LS
69-70	Linear					
71	Nonlinear	Z			3.048	IO - LS
72	Nonlinear	Z			3.048	IO - LS
73-74	Linear					
75	Nonlinear	Z			3.048	IO - LS
76	Inactive					
77-78	Linear					
79	Nonlinear	Z			3.048	IO - LS
80	Nonlinear	Z			3.048	IO - LS
81-82	Linear					
83	Nonlinear	Z			3.048	IO - LS
84	Nonlinear	Z			3.048	<= IO
85-98	Linear					
99	Nonlinear	Z	0	IO - LS	7.62	IO - LS
100	Nonlinear	Z	0	LS - CP	7.62	IO - LS
101	Nonlinear	Z	0	LS -	7.62	IO - LS

				CP		
102	Nonlinear	Z	0	IO - LS	7.62	IO - LS
117-119	Linear					
120	Nonlinear	Z	0	>= CP	7.62	LS - CP
121	Nonlinear	Z	0	IO - LS	7.62	<= IO
122	Nonlinear	Z	0	IO - LS	7.62	IO - LS
123	Nonlinear	Z	0	IO - LS	7.62	IO - LS
138-140	Linear					
141	Nonlinear	Z	0	IO - LS	7.62	IO - LS
142	Nonlinear	Z	0	<= IO	7.62	<= IO
143	Nonlinear	Z	0	IO - LS	7.62	IO - LS
144	Nonlinear	Z	0	IO - LS	7.62	IO - LS
159-161	Linear					
162	Nonlinear	Z	0	IO - LS	7.62	<= IO
163	Nonlinear	Z	0	IO - LS	7.62	IO - LS
164	Nonlinear	Z	0	IO - LS	7.62	IO - LS
165	Nonlinear	Z	0	IO - LS	7.62	<= IO
180-213	Linear					
214	Nonlinear	Z	0	IO - LS	7.62	<= IO
215	Nonlinear	Z	0	LS -	7.62	IO - LS

				CP		
216	Nonlinear	Z	0	LS - CP	7.62	IO - LS
217	Nonlinear	Z	0	IO - LS	7.62	<= IO
232-234	Linear					
235	Nonlinear	Z	0	>= CP	7.62	>= CP
236	Nonlinear	Z	0	LS - CP	7.62	IO - LS
237	Nonlinear	Z	0	IO - LS	7.62	IO - LS
238	Nonlinear	Z	0	IO - LS	7.62	IO - LS
253-255	Linear					
256	Nonlinear	Z	0	IO - LS	7.62	IO - LS
257	Nonlinear	Z	0	IO - LS	7.62	IO - LS
258	Nonlinear	Z	0	LS - CP	7.62	IO - LS
259	Nonlinear	Z	0	<= IO	7.62	<= IO
274-276	Linear					
277	Nonlinear	Z	0	IO - LS	7.62	<= IO
278	Nonlinear	Z	0	LS - CP	7.62	IO - LS
279	Nonlinear	Z	0	IO - LS	7.62	IO - LS
280	Nonlinear	Z	0	IO - LS	7.62	<= IO

295-312	Linear					
313	Nonlinear	Z	0	IO - LS	7.62	<= IO
314	Nonlinear	Z	0	IO - LS	7.62	<= IO
315	Nonlinear	Z	0	IO - LS	7.62	IO - LS
316	Nonlinear	Z			7.62	<= IO
331-333	Linear					
334	Nonlinear	Z	0	<= IO	7.62	<= IO
335	Nonlinear	Z	0	IO - LS	7.62	IO - LS
336	Nonlinear	Z	0	IO - LS	7.62	IO - LS
337	Nonlinear	Z			7.62	<= IO
352-354	Linear					

When base shear is 2380.605 KN during load step:173 displacement observed is 940.62mm.

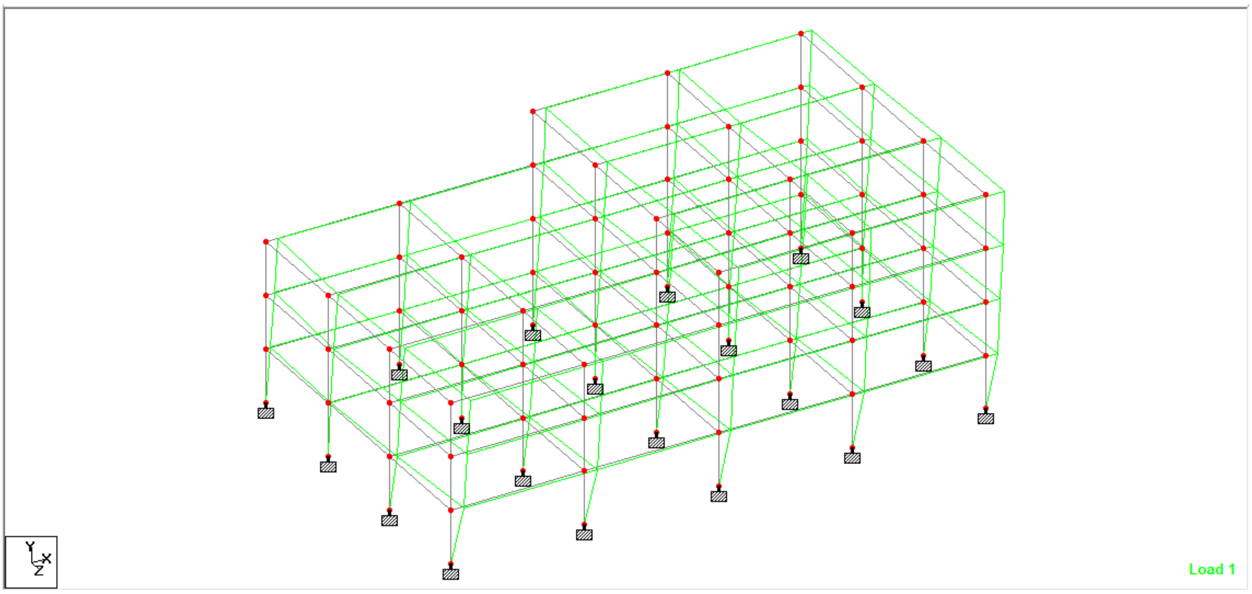


Fig 4. 15Displacement when base shear is 2380.605KN.

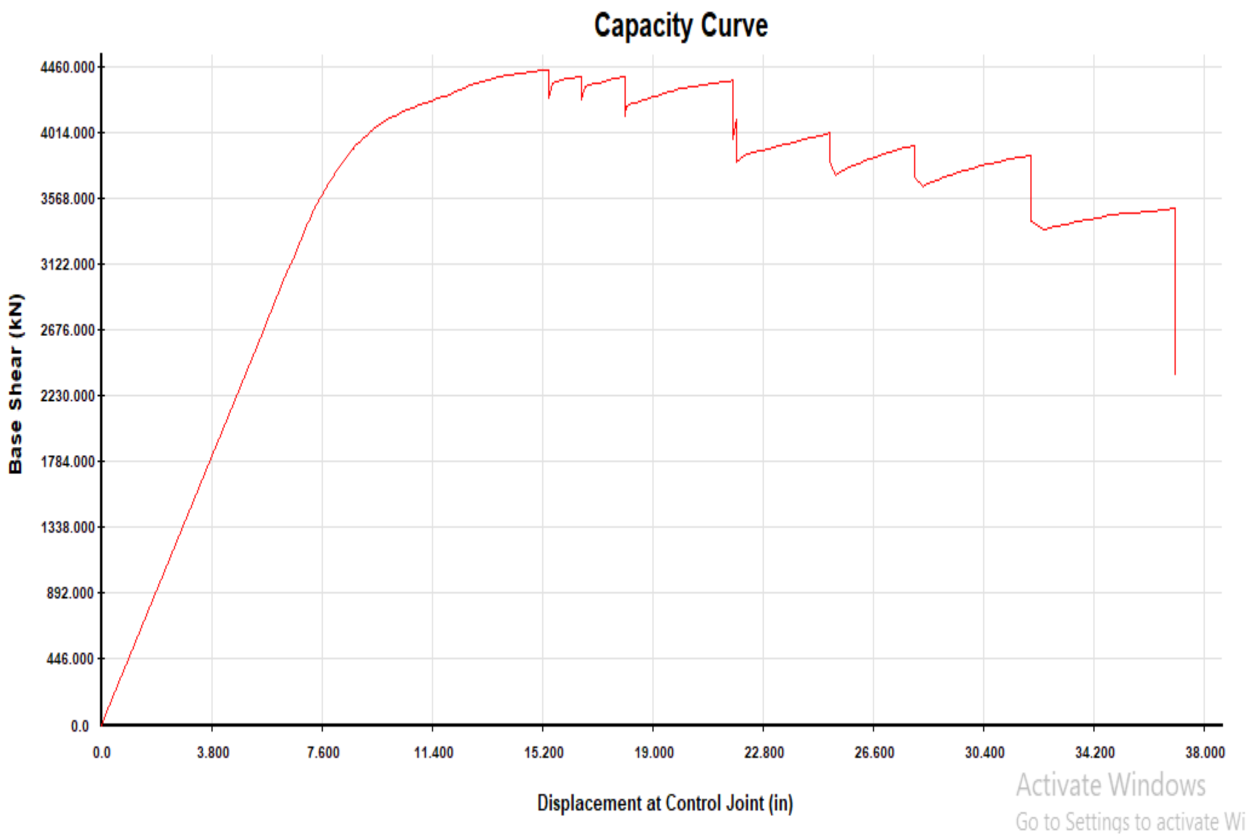


Fig 4. 16Capacity Curve of Moment Frame

In Table: 4.10values of base shear and displacement at that load are computed.

Table 4. 10Displacement and Base Shear values of 3D Frame without bracing

Load Step (%s)	Displacement mm	Base Shear kN
1	0	0
2	0.808	15.312
3	7.13	135.097
4	144.692	2741.101
5	147.934	2801.455
6	151.224	2862.703
7	154.514	2923.951
8	157.804	2985.199
9	161.094	3046.446

10	164.384	3107.694
11	167.673	3168.942
12	170.963	3230.19
13	174.253	3291.437
14	177.543	3352.685
15	180.833	3413.933
16	182.531	3444.557
17	185.251	3489.903
18	186.225	3504.198
19	189.332	3549.743
20	192.464	3595.679
21	193.538	3610.991
22	196.806	3655.731
23	199.15	3684.355
24	202.848	3728.198
25	205.432	3758.822
26	206.733	3774.134
27	209.303	3804.348
28	210.631	3819.66
29	211.905	3834.158
30	213.256	3848.976
31	216.133	3879.285
32	219.162	3909.165
33	220.858	3924.108
34	222.596	3938.632

35	224.414	3953.515
36	226.248	3968.217
37	228.182	3982.871
38	232.332	4012.994
39	234.42	4027.077
40	236.714	4041.484
41	241.507	4071.479
42	246.981	4102.103
43	249.689	4115.408
44	252.744	4130.424
45	256.908	4145.736
46	260.454	4158.202
47	264.725	4173.01
48	269.028	4187.92
49	273.517	4203.211
50	278.338	4218.078
51	283.497	4232.067
52	288.844	4246.098
53	294.438	4260.757
54	300.267	4276.034
55	306.11	4291.346
56	310.417	4306.658
57	314.467	4320.344
58	318.885	4335.204
59	323.421	4350.462

60	328.184	4365.774
61	334.095	4380.597
62	340.166	4392.605
63	347.582	4405.885
64	356.704	4419.854
65	367.204	4432.628
66	378.62	4444.883
67	385.052	4451.747
68	392.382	4459.365
69	392.382	4260.879
70	395.584	4370.19
71	399.251	4377.846
72	403.298	4385.502
73	407.613	4393.158
74	412.489	4400.814
75	417.487	4408.47
76	420.047	4412.298
77	420.047	4258.753
78	423.768	4347.573
79	427.989	4355.229
80	432.254	4362.885
81	436.435	4370.541
82	440.62	4378.197
83	444.81	4385.853
84	449.011	4393.509

85	458.212	4408.668
86	458.212	4146.124
87	459.72	4212.074
88	462.448	4219.73
89	465.175	4227.386
90	468.056	4235.042
91	471.057	4242.698
92	474.152	4250.354
93	477.298	4258.01
94	480.443	4265.666
95	483.589	4273.322
96	486.735	4280.978
97	489.971	4288.634
98	493.208	4296.29
99	496.444	4303.946
100	499.862	4311.602
101	503.344	4319.258
102	507.044	4326.914
103	512.151	4334.571
104	515.154	4338.399
105	518.156	4342.227
106	521.172	4346.055
107	524.187	4349.883
108	527.213	4353.711
109	530.274	4357.539

110	533.361	4361.367
111	536.519	4365.195
112	539.676	4369.023
113	542.898	4372.851
114	546.187	4376.679
115	549.511	4380.507
116	552.84	4384.335
117	552.84	3990.612
118	556.61	4121.995
119	556.61	3825.046
120	562.342	3876.098
121	569.874	3891.4
122	577.404	3906.697
123	584.932	3921.99
124	592.458	3937.279
125	599.982	3952.565
126	607.504	3967.846
127	615.024	3983.124
128	622.542	3998.397
129	630.059	4013.667
130	637.573	4028.933
131	637.573	3832.553
132	643.105	3746.222
133	646.684	3761.386
134	650.596	3776.558

135	654.683	3791.78
136	658.926	3806.961
137	663.776	3822.419
138	668.59	3837.762
139	673.369	3852.992
140	678.66	3868.233
141	684.303	3883.416
142	691.047	3898.729
143	698.052	3913.974
144	705.238	3929.301
145	712.35	3944.471
146	712.35	3732.837
147	719.28	3666.623
148	723.292	3681.884
149	727.733	3697.056
150	732.27	3712.259
151	738.141	3727.44
152	744.231	3742.685
153	750.403	3757.877
154	756.636	3773.087
155	762.912	3788.252
156	769.63	3803.427
157	776.758	3818.591
158	786.031	3833.794
159	795.283	3848.961

160	804.574	3864.175
161	813.831	3879.335
162	813.831	3424.072
163	826.234	3376.209
164	835.707	3391.919
165	845.171	3407.613
166	854.556	3423.176
167	863.85	3438.589
168	873.056	3453.855
169	882.594	3469.039
170	899.454	3484.259
171	920.036	3499.603
172	940.62	3514.948
173	940.62	2380.605

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Following table 4.11contain Status of hinge formation in the frame without bracing after completing pushover analysis.

Table 4. 11Hinge Status when Base Shear is 2380.605KN

Beam	Status	Dir (Local)	Section m	Status	Section m	Status
1	Inactive					
2	Nonlinear	Z			3.048	IO - LS
3-5	Inactive					
6	Nonlinear	Z	0	IO - LS	3.048	IO - LS
7-8	Inactive					

9	Nonlinear	Z			3.048	>= CP
10-13	Inactive					
14	Nonlinear	Z	0	IO - LS	3.048	IO - LS
15-16	Inactive					
17	Nonlinear	Z			3.048	IO - LS
18	Nonlinear	Z			3.048	IO - LS
19-20	Inactive					
21-23	Linear					
24	Nonlinear	Z	0	IO - LS	7.62	IO - LS
25	Nonlinear	Z	0	<= IO		
26	Nonlinear	Z	0	<= IO	7.62	<= IO
27	Nonlinear	Z	0	IO - LS	7.62	IO - LS
28-30	Linear					
31	Nonlinear	Z	0	>= CP	7.62	>= CP
32-33	Linear					
34	Nonlinear	Z	0	IO - LS	7.62	IO - LS
35-37	Linear					
38	Nonlinear	Z	0	LS - CP	7.62	IO - LS
39	Linear					
40	Nonlinear	Z			7.62	<= IO
41	Nonlinear	Z	0	LS - CP	7.62	IO - LS
42-44	Linear					
45	Nonlinear	Z	0	IO - LS	7.62	IO - LS
46	Nonlinear	Z			7.62	<= IO

47	Nonlinear	Z	0	<= IO	7.62	IO - LS
48	Nonlinear	Z	0	IO - LS	7.62	IO - LS
49-66	Linear					
67	Nonlinear	Z			3.048	IO - LS
68	Nonlinear	Z			3.048	IO - LS
69-70	Linear					
71	Nonlinear	Z			3.048	IO - LS
72	Nonlinear	Z			3.048	IO - LS
73-74	Linear					
75	Nonlinear	Z			3.048	IO - LS
76	Inactive					
77-78	Linear					
79	Nonlinear	Z			3.048	IO - LS
80	Nonlinear	Z			3.048	IO - LS
81-82	Linear					
83	Nonlinear	Z			3.048	IO - LS
84	Nonlinear	Z			3.048	<= IO
85-98	Linear					
99	Nonlinear	Z	0	IO - LS	7.62	IO - LS
100	Nonlinear	Z	0	LS - CP	7.62	IO - LS
101	Nonlinear	Z	0	LS - CP	7.62	IO - LS
102	Nonlinear	Z	0	IO - LS	7.62	IO - LS
117-119	Linear					

120	Nonlinear	Z	0	>= CP	7.62	LS - CP
121	Nonlinear	Z	0	IO - LS	7.62	<= IO
122	Nonlinear	Z	0	IO - LS	7.62	IO - LS
123	Nonlinear	Z	0	IO - LS	7.62	IO - LS
138-140	Linear					
141	Nonlinear	Z	0	IO - LS	7.62	IO - LS
142	Nonlinear	Z	0	<= IO	7.62	<= IO
143	Nonlinear	Z	0	IO - LS	7.62	IO - LS
144	Nonlinear	Z	0	IO - LS	7.62	IO - LS
159-161	Linear					
162	Nonlinear	Z	0	IO - LS	7.62	<= IO
163	Nonlinear	Z	0	IO - LS	7.62	IO - LS
164	Nonlinear	Z	0	IO - LS	7.62	IO - LS
165	Nonlinear	Z	0	IO - LS	7.62	<= IO
180-213	Linear					
214	Nonlinear	Z	0	IO - LS	7.62	<= IO
215	Nonlinear	Z	0	LS - CP	7.62	IO - LS
216	Nonlinear	Z	0	LS - CP	7.62	IO - LS
217	Nonlinear	Z	0	IO - LS	7.62	<= IO
232-234	Linear					

235	Nonlinear	Z	0	>= CP	7.62	>= CP
236	Nonlinear	Z	0	LS - CP	7.62	IO - LS
237	Nonlinear	Z	0	IO - LS	7.62	IO - LS
238	Nonlinear	Z	0	IO - LS	7.62	IO - LS
253-255	Linear					
256	Nonlinear	Z	0	IO - LS	7.62	IO - LS
257	Nonlinear	Z	0	IO - LS	7.62	IO - LS
258	Nonlinear	Z	0	LS - CP	7.62	IO - LS
259	Nonlinear	Z	0	<= IO	7.62	<= IO
274-276	Linear					
277	Nonlinear	Z	0	IO - LS	7.62	<= IO
278	Nonlinear	Z	0	LS - CP	7.62	IO - LS
279	Nonlinear	Z	0	IO - LS	7.62	IO - LS
280	Nonlinear	Z	0	IO - LS	7.62	<= IO
295-312	Linear					
313	Nonlinear	Z	0	IO - LS	7.62	<= IO
314	Nonlinear	Z	0	IO - LS	7.62	<= IO
315	Nonlinear	Z	0	IO - LS	7.62	IO - LS
316	Nonlinear	Z			7.62	<= IO
331-333	Linear					

334	Nonlinear	Z	0	<= IO	7.62	<= IO
335	Nonlinear	Z	0	IO - LS	7.62	IO - LS
336	Nonlinear	Z	0	IO - LS	7.62	IO - LS
337	Nonlinear	Z			7.62	<= IO
352-354	Linear					

b.Steel Frame with External Bracing

G+3 storey frame with analyzed by static nonlinear process, frame performed linearly up to the base shear 137.70KN. When base shear is 3879.86KN column 6,7,14 and 15 are in IO performance level as green colored plastic hinges developed in it as shown in fig54. When base shear reached the value 5161.37KN column10 and 11 is in IO – LS performance level as shown in fig 55 column 10 and 11 reached LS-CP performance level at base shear 5406.2794KN and in complete CP level when base shear 5545.04KN as shown in fig 56. Bracing provided started failing when base shear 5636.119KN as shown in fig 57.It is observed that due to external bracings lateral load carrying capacity of structure is increased but displacement is also more which laid to failure of structure. After that base shear redistributed up to the push load stem 231 and the Maximum columns of basement were failed at base shear 6039.11KN. After which entire structure will collapses. Capacity curve obtained for this frame is as shown in fig 4.18.

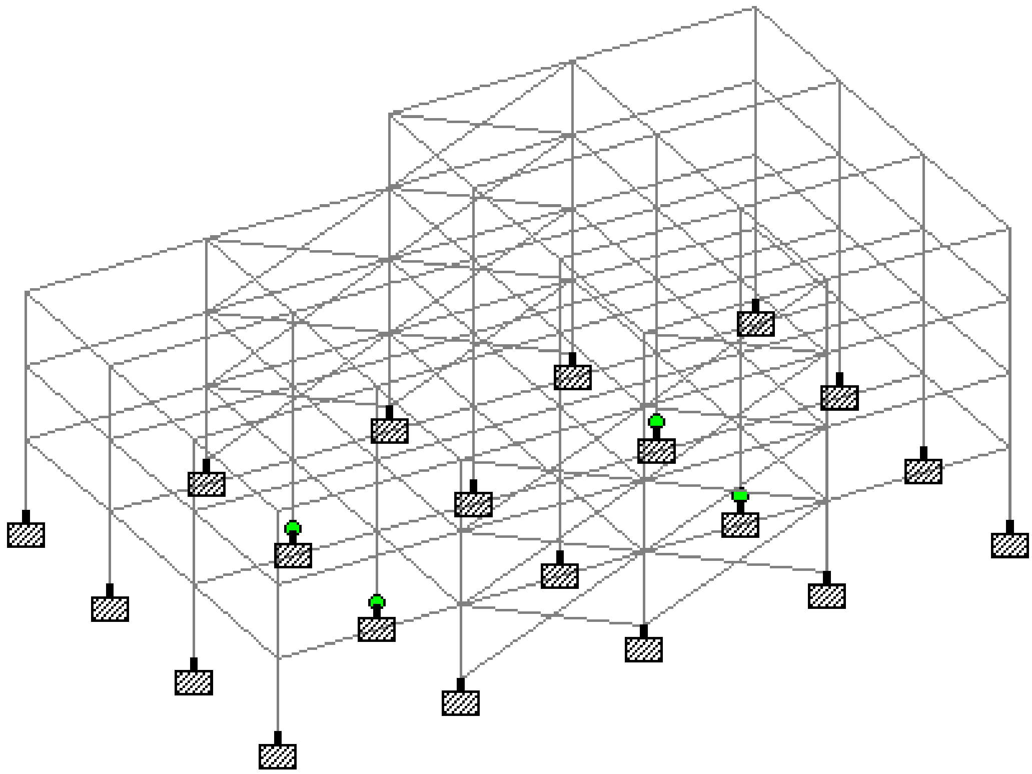

Fig 4. 17Members of Externally Braced Steel Frame in IO

When base shear is 3879.864 KN during load step: 4, plastic hinge status is computed in Table 4.12

Table 4. 12Hinge Location Status when Base Shear is 3879.864KN

Beam	Status	Dir (Local)	Section m	Status	Section m	Status	Section m	Status
1-5	Linear							
6	Nonlinear	Z					3.048	<= IO
7	Nonlinear	Z					3.048	<= IO
8-13	Linear							
14	Nonlinear	Z					3.048	<= IO
15	Nonlinear	Z					3.048	<= IO
16-382	Linear							

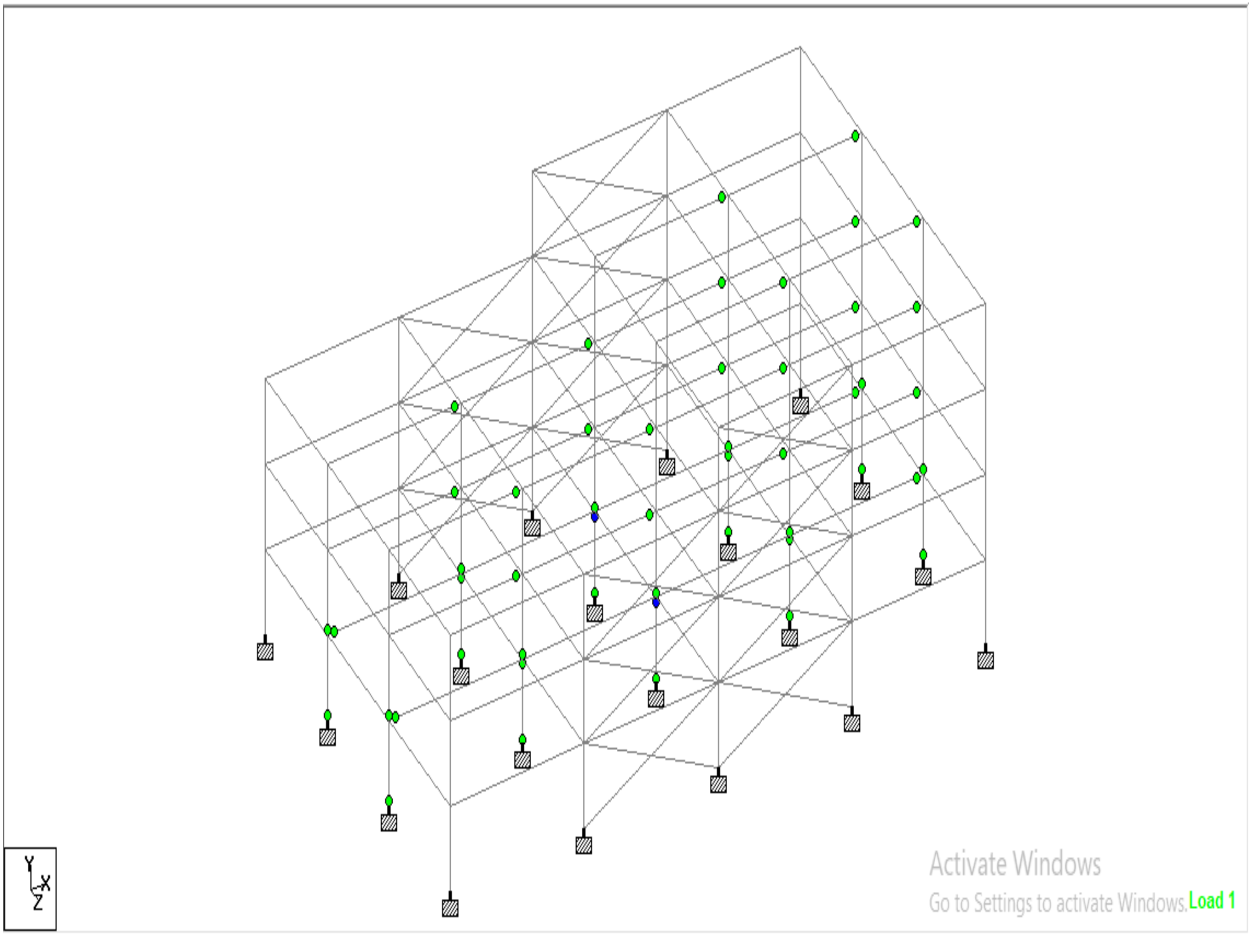

Fig 4. 18Members of Externally Braced Steel Frame in IO-LS

Table4.13 shows hinge formation status on load step 48 when base shear value is 5161.377KN.

Table 4. 13Hinge location Status when Base Shear is 5161.377KN

Beam	Status	Dir (Local)	Section ft	Status	Section ft	Status
1	Linear					
2	Nonlinear	Z			10	<= IO
3	Nonlinear	Z			10	<= IO
4-5	Linear					
6	Nonlinear	Z	0	<= IO	10	<= IO
7	Nonlinear	Z	0	<= IO	10	<= IO
8-9	Linear					

10	Nonlinear	Z	0	IO - LS	10	<= IO
11	Nonlinear	Z	0	IO - LS	10	<= IO
12-13	Linear					
14	Nonlinear	Z	0	<= IO	10	<= IO
15	Nonlinear	Z	0	<= IO	10	<= IO
16-17	Linear					
18	Nonlinear	Z			10	<= IO
19	Nonlinear	Z			10	<= IO
20-24	Linear					
25	Nonlinear	Z	0	<= IO		
26	Nonlinear	Z	0	<= IO		
27-45	Linear					
46	Nonlinear	Z			25	<= IO
47	Nonlinear	Z			25	<= IO
48-66	Linear					
67	Nonlinear	Z			10	<= IO
68	Nonlinear	Z			10	<= IO
69-70	Linear					
71	Nonlinear	Z			10	<= IO
72	Nonlinear	Z			10	<= IO
73-74	Linear					
75	Nonlinear	Z			10	<= IO
76	Nonlinear	Z			10	<= IO
77-78	Linear					

79	Nonlinear	Z			10	<= IO
80	Nonlinear	Z			10	<= IO
81-82	Linear					
83	Nonlinear	Z			10	<= IO
84	Nonlinear	Z			10	<= IO
85-99	Linear					
100	Nonlinear	Z			25	<= IO
101	Nonlinear	Z			25	<= IO
102	Linear					
117-120	Linear					
121	Nonlinear	Z			25	<= IO
122	Nonlinear	Z			25	<= IO
123-141	Linear					
142	Nonlinear	Z			25	<= IO
143	Nonlinear	Z			25	<= IO
144-162	Linear					
163	Nonlinear	Z			25	<= IO
164	Nonlinear	Z			25	<= IO
165-214	Linear					
215	Nonlinear	Z			25	<= IO
216	Nonlinear	Z			25	<= IO
217-235	Linear					
236	Nonlinear	Z			25	<= IO
237	Nonlinear	Z			25	<= IO

238-256	Linear					
257	Nonlinear	Z			25	<= IO
258	Nonlinear	Z			25	<= IO
259-277	Linear					
278	Nonlinear	Z			25	<= IO
279	Nonlinear	Z			25	<= IO
280-313	Linear					
314	Nonlinear	Z			25	<= IO
315	Nonlinear	Z			25	<= IO
316-334	Linear					
335	Nonlinear	Z			25	<= IO
336	Nonlinear	Z			25	<= IO
337-382	Linear					

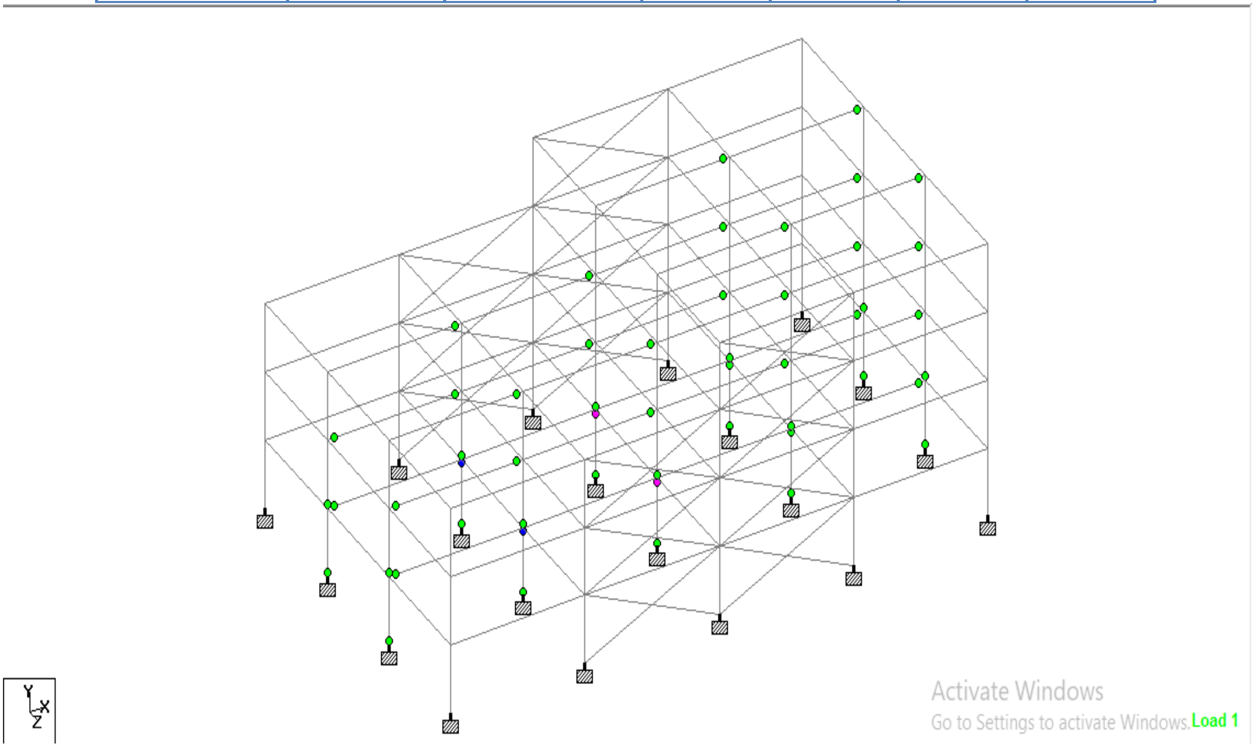

Fig 4. 19Members of Externally Braced Steel Frame in LS-CP

Table 4.14 shows plastic hinge location status of beams in the frame with external bracings when base shear redistributed and its value is 5391.19KN at load step: 63.

Table 4. 14Hinge Location Status when redistributed Base Shear is 5391.19KN

Beam	Status	Dir (Local)	Section ft	Status	Section ft	Status
1	Linear					
2	Nonlinear	Z			10	<= IO
3	Nonlinear	Z			10	<= IO
4-5	Linear					
5	Linear					
6	Nonlinear	Z	0	IO – LS	10	<= IO
7	Nonlinear	Z	0	IO – LS	10	<= IO
8-9	Linear					
10	Nonlinear	Z	0	LS – CP	10	<= IO
11	Nonlinear	Z	0	LS – CP	10	<= IO
12-13	Linear					
14	Nonlinear	Z	0	<= IO	10	<= IO
15	Nonlinear	Z	0	<= IO	10	<= IO
16-17	Linear					
18	Nonlinear	Z			10	<= IO
19	Nonlinear	Z			10	<= IO
20-24	Linear					
25	Nonlinear	Z	0	<= IO		
26	Nonlinear	Z	0	<= IO		
27-45	Linear					

46	Nonlinear	Z			25	<= IO
47	Nonlinear	Z			25	<= IO
48-66	Linear					
67	Nonlinear	Z			10	<= IO
68	Nonlinear	Z			10	<= IO
69-70	Linear					
71	Nonlinear	Z			10	<= IO
72	Nonlinear	Z			10	<= IO
73-74	Linear					
75	Nonlinear	Z			10	<= IO
76	Nonlinear	Z			10	<= IO
77-78	Linear					
79	Nonlinear	Z			10	<= IO
80	Nonlinear	Z			10	<= IO
81-82	Linear					
83	Nonlinear	Z			10	<= IO
84	Nonlinear	Z			10	<= IO
85-99	Linear					
100	Nonlinear	Z	0	<= IO	25	<= IO
101	Nonlinear	Z	0	<= IO	25	<= IO
102-120	Linear					
121	Nonlinear	Z			25	<= IO
122	Nonlinear	Z			25	<= IO

123-141	Linear					
142	Nonlinear	Z			25	<= IO
143	Nonlinear	Z			25	<= IO
144-162	Linear					
163	Nonlinear	Z			25	<= IO
164	Nonlinear	Z			25	<= IO
165-214	Linear					
215	Nonlinear	Z			25	<= IO
216	Nonlinear	Z			25	<= IO
217-235	Linear					
236	Nonlinear	Z			25	<= IO
237	Nonlinear	Z			25	<= IO
238-256	Linear					
257	Nonlinear	Z			25	<= IO
258	Nonlinear	Z			25	<= IO
259	Linear					
274-277	Linear					
278	Nonlinear	Z			25	<= IO
279	Nonlinear	Z			25	<= IO

280-313	Linear					
314	Nonlinear	Z			25	<= IO
315	Nonlinear	Z			25	<= IO
316-334	Linear					
335	Nonlinear	Z			25	<= IO
336	Nonlinear	Z			25	<= IO
337-382	Linear					

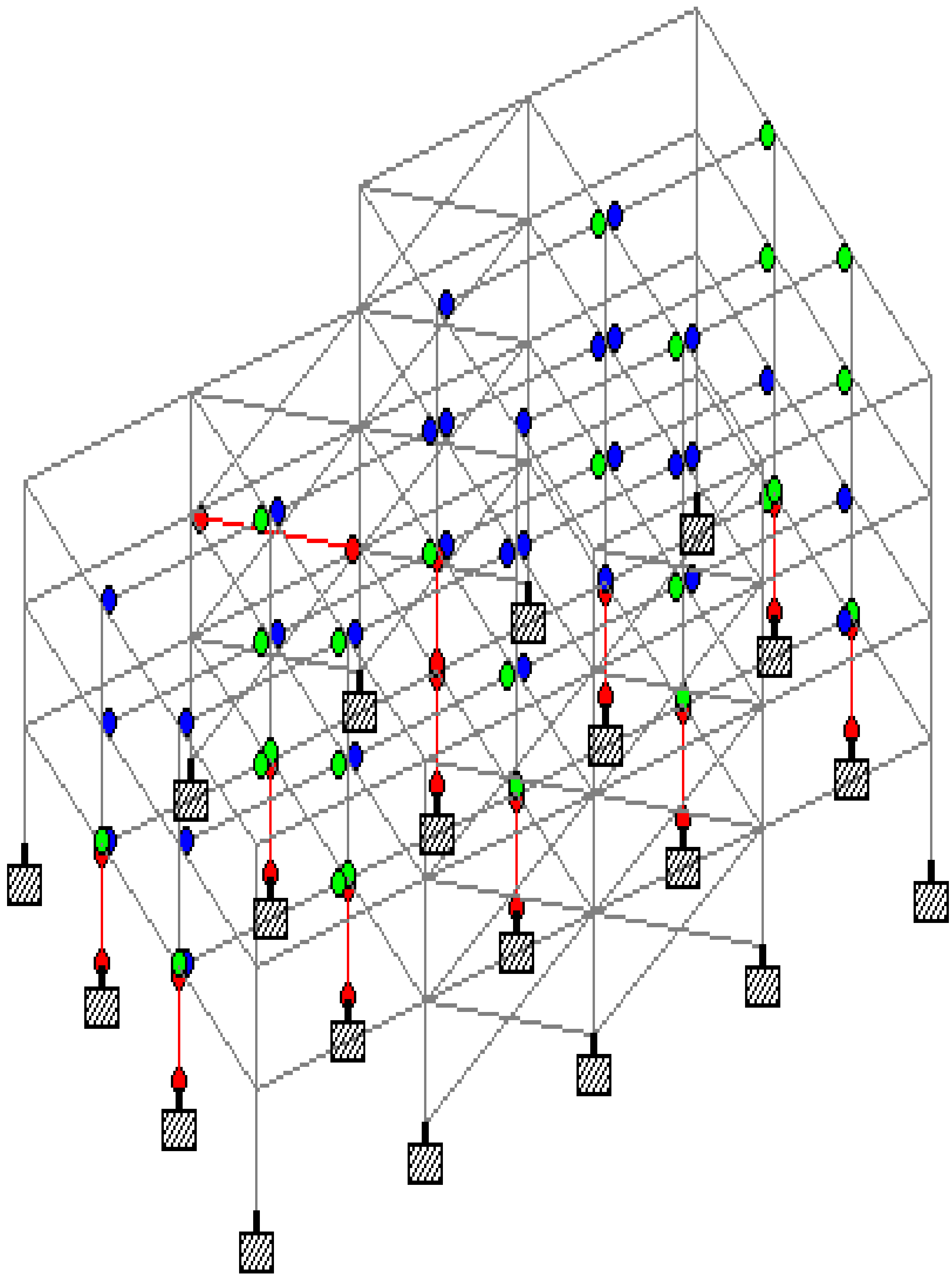

Fig 4. 20Members of Externally Braced Steel Frame Failed in Pushover Analysis

At the load step:231 base shear applied is 6039.118KN and displacement recorded by software is 1202.272mm as shown in below Fig: 4.21

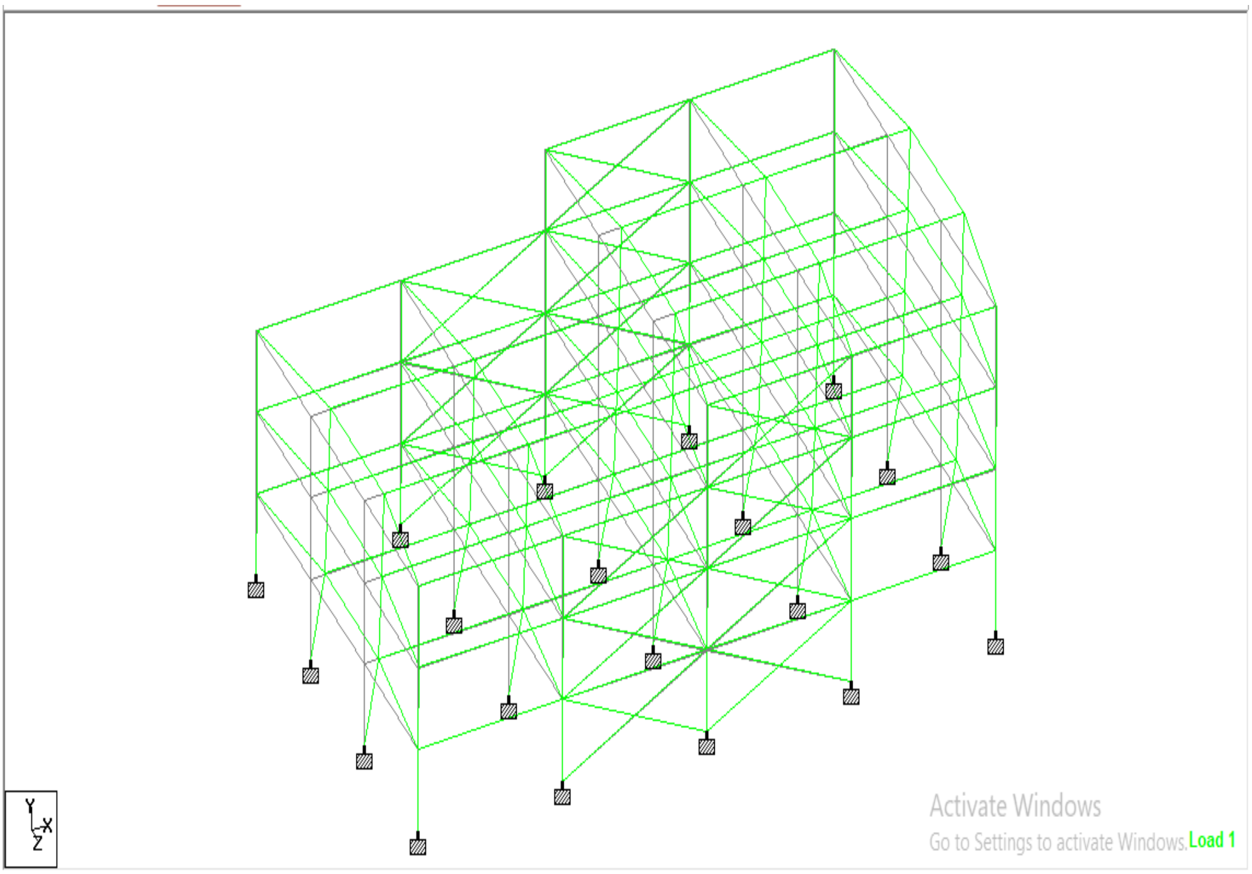

Fig 4. 21Displacement in Externally Braced Steel Frame when Base Shear is 6039.118KN

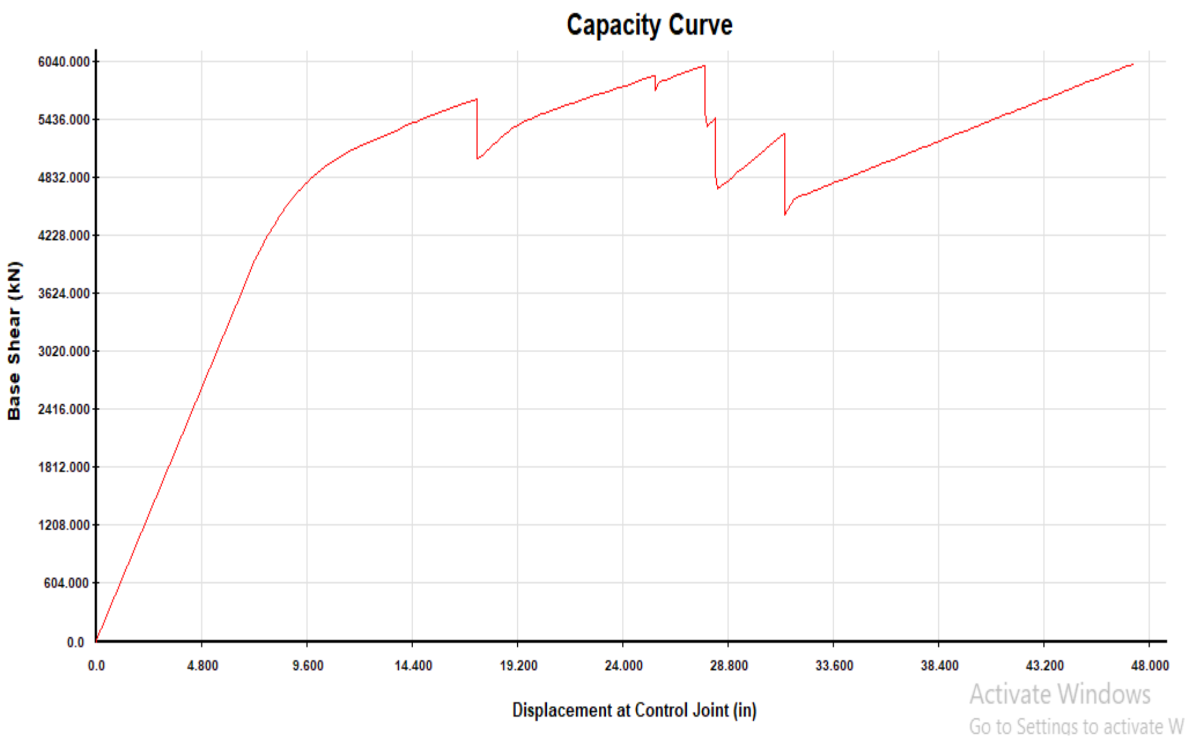

Fig 4. 22Capacity Curve for Externally Braced Steel Frame

Table 4.15Shows the displacement obsered during respective load step and base shear in externally braced 3D steel frame.

Table 4. 15Displacement and Base Shear.

Load Step (%s)	Displacement mm	Base Shear kip
1	0	0
2	0.712	3.485
3	6.325	30.958
4	178.265	872.228
5	181.325	886.05
6	183.8	896.277
7	186.432	906.424
8	189.144	916.879
9	191.856	927.334
10	194.568	937.789
11	196.455	944.758
12	199.35	954.928
13	202.361	965.383
14	203.395	968.757
15	206.572	979.085
16	209.787	989.54
17	213.002	999.995
18	216.217	1010.45
19	217.335	1013.935
20	219.577	1020.765

21	220.804	1024.123
22	223.354	1030.942
23	225.969	1037.764
24	227.369	1041.249
25	230.113	1048.061
26	232.92	1055.031
27	234.395	1058.516
28	235.876	1061.844
29	238.915	1068.651
30	242.114	1075.621
31	245.324	1082.404
32	248.623	1089.374
33	252.25	1096.344
34	256.2	1102.709
35	260.537	1109.222
36	265.179	1116.192
37	267.741	1119.677
38	272.686	1126.257
39	275.305	1129.742
40	277.925	1133.227
41	280.544	1136.712
42	283.163	1140.197
43	285.782	1143.682
44	288.461	1147.167
45	291.4	1150.575

46	294.126	1153.692
47	297.417	1157.129
48	300.489	1160.324
49	304.42	1163.794
50	307.774	1166.754
51	311.722	1170.239
52	315.67	1173.724
53	319.619	1177.209
54	323.447	1180.694
55	327.397	1184.288
56	331.226	1187.77
57	335.274	1191.255
58	339.116	1194.563
59	343.165	1198.048
60	347.213	1201.533
61	351.261	1205.018
62	355.31	1208.503
63	359.471	1211.988
64	363.522	1215.38
65	368.092	1218.865
66	372.262	1222.045
67	376.831	1225.53
68	381.401	1229.015
69	385.79	1232.5
70	390.352	1236.122

71	394.741	1239.607
72	399.131	1243.092
73	403.52	1246.577
74	408.07	1250.062
75	412.463	1253.427
76	417.21	1256.912
77	422.013	1260.256
78	426.767	1263.565
79	431.779	1267.05
80	436.781	1270.528
81	441.794	1274.013
82	441.794	1136.805
83	445.325	1137.671
84	448.882	1144.801
85	452.405	1151.865
86	455.896	1158.863
87	459.356	1165.799
88	461.215	1169.25
89	465.062	1176.368
90	468.87	1183.416
91	471.7	1188.652
92	474.648	1193.879
93	477.308	1198.236
94	480.216	1202.592
95	483.189	1206.948

96	485.905	1210.433
97	489.03	1213.918
98	492.233	1217.403
99	495.667	1220.888
100	498.392	1223.502
101	501.193	1226.116
102	504.113	1228.729
103	507.033	1231.343
104	510.043	1233.957
105	513.181	1236.571
106	516.419	1239.184
107	519.71	1241.798
108	523.188	1244.412
109	526.74	1247.025
110	536.319	1253.899
111	544.685	1259.88
112	553.022	1265.84
113	561.329	1271.78
114	569.858	1277.698
115	578.75	1283.593
116	587.574	1289.459
117	596.352	1295.295
118	605.086	1301.101
119	613.776	1306.877
120	622.422	1312.626

121	631.026	1318.345
122	639.588	1324.037
123	648.108	1329.701
124	648.108	1293.42
125	651.236	1312.289
126	653.98	1314.902
127	657.233	1317.516
128	660.699	1320.13
129	664.165	1322.744
130	667.632	1325.357
131	671.098	1327.971
132	675.745	1331.427
133	680.385	1334.878
134	685.188	1338.445
135	689.948	1341.979
136	694.666	1345.482
137	699.344	1348.955
138	705.099	1353.016
139	705.099	1244.291
140	708.238	1210.373
141	711.4	1217.412
142	714.548	1224.421
143	717.684	1231.403
144	717.684	1096.859
145	720.32	1063.002

146	724.74	1070.543
147	729.132	1078.035
148	733.496	1085.48
149	737.832	1092.879
150	742.142	1100.232
151	746.426	1107.54
152	750.683	1114.805
153	754.916	1122.026
154	759.123	1129.204
155	763.306	1136.341
156	767.466	1143.437
157	771.601	1150.493
158	775.713	1157.509
159	779.803	1164.486
160	783.87	1171.425
161	787.949	1178.359
162	792.373	1185.28
163	795.054	1188.734
164	797.979	1192.204
165	797.979	1003.914
166	808.968	1040.845
167	815.015	1045.716
168	821.062	1050.587
169	827.11	1055.458
170	833.157	1060.329

171	839.205	1065.2
172	845.252	1070.072
173	851.3	1074.943
174	857.348	1079.815
175	863.396	1084.686
176	869.444	1089.558
177	875.493	1094.43
178	881.541	1099.302
179	887.59	1104.174
180	893.638	1109.046
181	899.687	1113.918
182	905.736	1118.791
183	911.785	1123.663
184	917.834	1128.535
185	923.883	1133.408
186	929.932	1138.281
187	935.982	1143.153
188	942.031	1148.026
189	948.081	1152.899
190	954.131	1157.772
191	960.181	1162.645
192	966.231	1167.519
193	972.281	1172.392
194	978.331	1177.265
195	984.381	1182.139

196	990.432	1187.012
197	996.483	1191.886
198	1002.533	1196.76
199	1008.584	1201.634
200	1014.635	1206.508
201	1020.686	1211.382
202	1026.737	1216.256
203	1032.789	1221.13
204	1038.84	1226.005
205	1044.892	1230.879
206	1050.943	1235.754
207	1056.995	1240.628
208	1063.047	1245.503
209	1069.099	1250.378
210	1075.151	1255.253
211	1081.203	1260.128
212	1087.256	1265.003
213	1093.308	1269.878
214	1099.361	1274.753
215	1105.413	1279.629
216	1111.466	1284.504
217	1117.519	1289.38
218	1123.572	1294.256
219	1129.625	1299.131
220	1135.679	1304.007

221	1141.732	1308.883
222	1147.786	1313.759
223	1153.839	1318.635
224	1159.893	1323.511
225	1165.947	1328.388
226	1172.001	1333.264
227	1178.055	1338.141
228	1184.109	1343.017
229	1190.163	1347.894
230	1196.218	1352.771
231	1202.272	1357.648

C.Steel Frame with Optimum Bracing

After the results of above two frames third frame is designed in such way that internal bracings were provided at position to avoid the failure of members observed in first and second frame case. For that purpose different position of the bracing tried and the frame represented in this paper with the optimal position of bracing is with the most accurate collapse prevention results. Column 5,8,912,13 and 16 in G+3 internally braced steel frame is in IO performance level ate base shear 3536KN as shown in fig 60. Column 27 and 31reched the IO-LS performance level when base shear is 4517.66 KN as shown in fig 4.24. Column 10 and 11 as shown in fig 4.26 are I LS-CP level when base shear 5128.46KN. Same columns reached CP level first in entire structure at base shear 5222.86KN. In this design, it is observed that base shear carrying capacity of structure is increased but displacement of structure is less as compare to externally braced frame. Hence structure prevented from collapse. After completing all push load step it is observed that no member failed only column 10 and 11 are in CP level but structure is safe on remaining columns. Capacity curve is as shown in fig 4.27

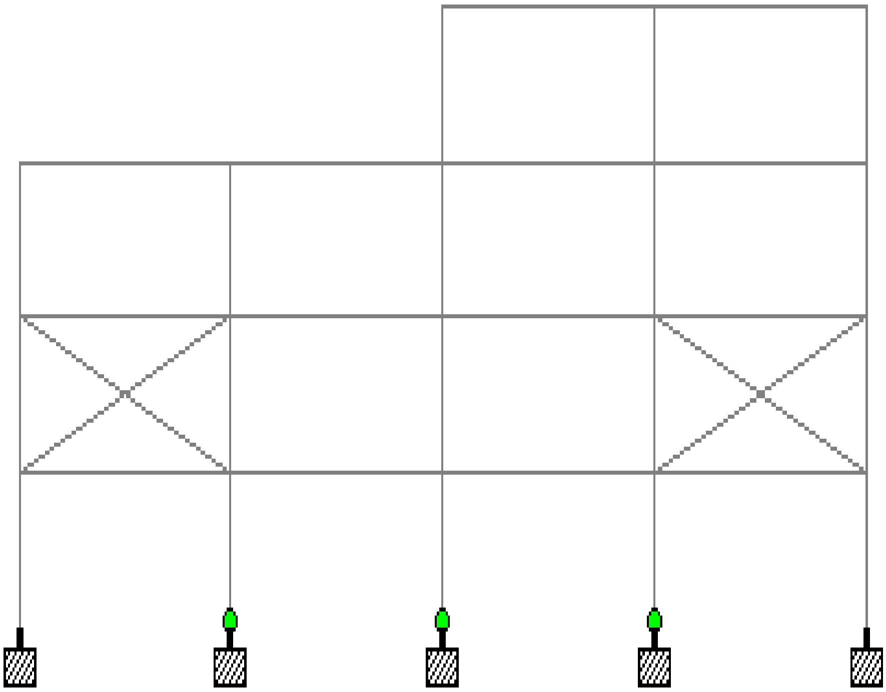

Fig 4. 23Members of Internally Braced Steel Frame in IO

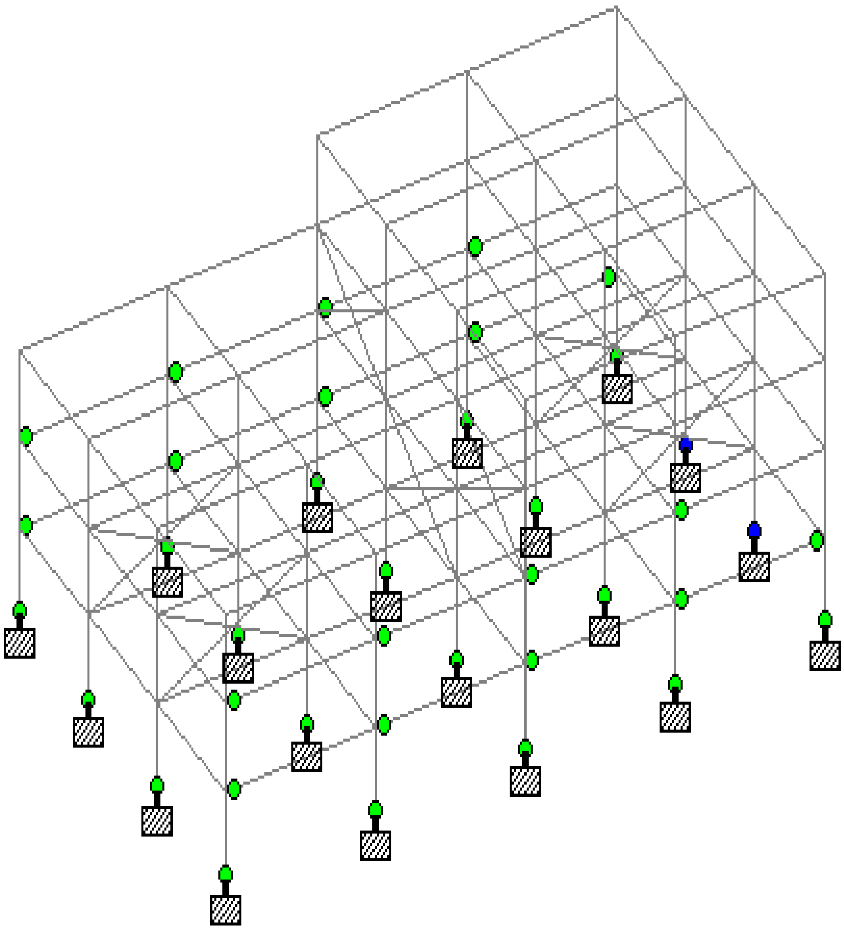

Fig 4. 24Members of Externally Braced Steel Frame in IO-LS

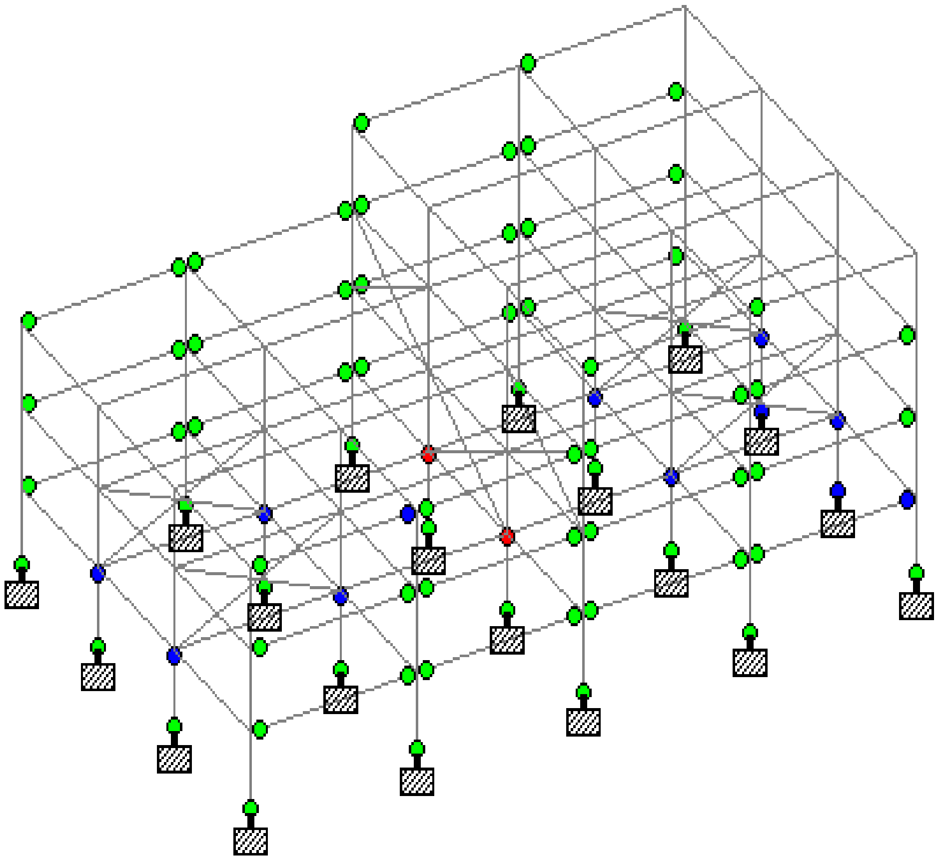

Fig 4. 25Members of Externally Braced Steel Frame in LS-CP Performance Level

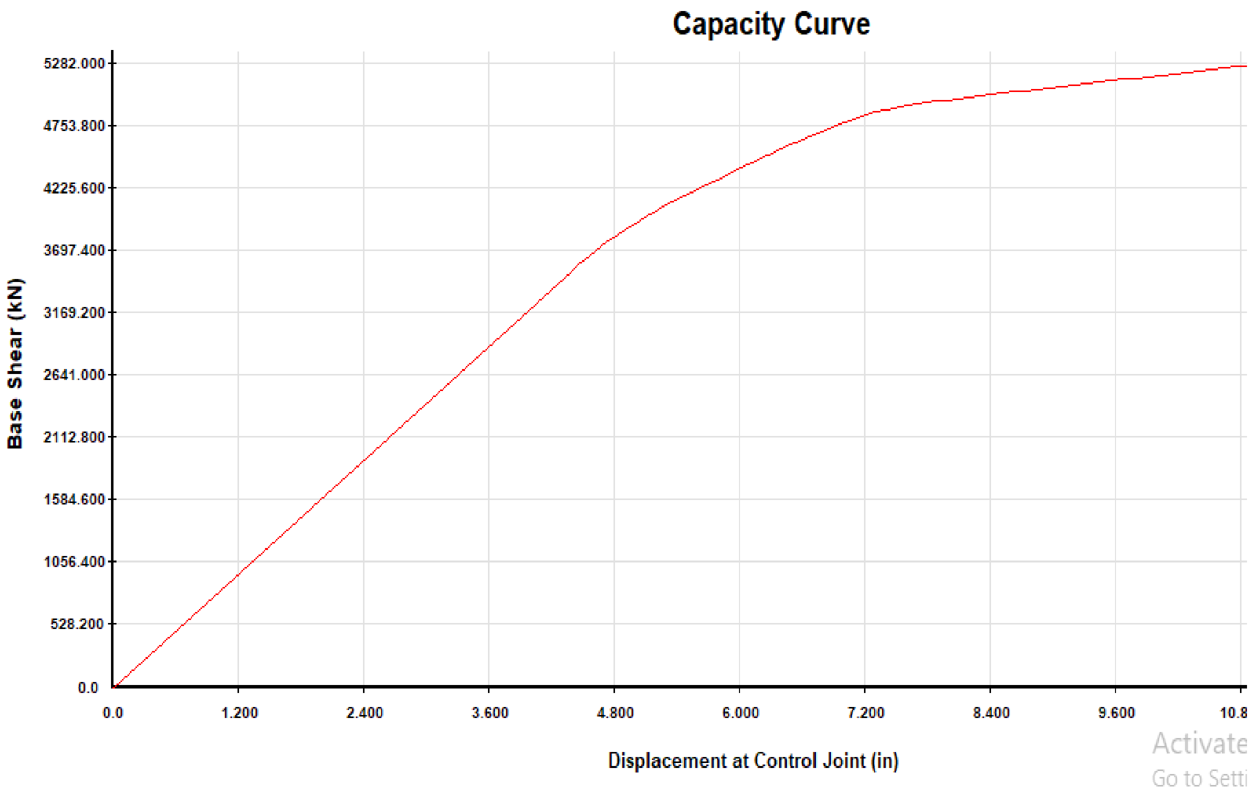

Fig 4. 26Capacity Curve for Externally Braced Steel Frame

CHAPTER 5
CONCLUSION

This thesis presented and documented performance based seismic analysis for steel frames. The concept of performance based seismic design was successfully implemented by nonlinear static analysis by applying incremental lateral loads on braced and non braced steel frames. The performance criteria suggested by FEMA 356 can be successfully implemented in PBSD pushover analysis method by using STAAD Pro. Advanced. Maximum members of moment frame reaches to Collapse prevention level and ultimately fails under the incremental push loads. This leads the collapse of entire steel frame during the earthquake. The Shear capacity of the structure can be increased by introducing external steel bracings in the structural system. But under the incremental lateral loads bracing also fail. This leads to the maximum members to be in CP level and causes failure of structural members during earthquake. To avoid this position of the bracing can be optimized by using pushover analysis by identifying which members are failing after incremental lateral load and identifying the position of bracing which prevents the failure of these members. Such optimal position of bracing saves the structure during earthquake. It is concluded in this research that such braced steel frame at optimal position increases the shear capacity of structure and performs well, maximum in LS level. No collapse of member is observed in this frame after incremental lateral loads. Pushover analysis is successfully implemented to study non linear behavior of structure under earthquake loading.

CHAPTER 6

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APPENDICES

2D Moment Resisting Frame

```
****************************************************
* *
* STAAD.Pro CONNECT Edition *
* Version 22.04.00.40 *
* Proprietary Program of *
* Bentley Systems, Inc. *
* Date= MAY 21, 2020 *
* Time= 15:46:58 *
* *
* Licensed to: Shri Vithal Education & Research *
****************************************************
1. STAAD SPACE PLANE
INPUT FILE: G:\MTech_Pr\Ex1_2D\2D_Ex1.STD
2. START JOB INFORMATION
3. ENGINEER DATE 07-OCT-05
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. UNIT INCHES KIP
7. JOINT COORDINATES
8. 1 0 0 0; 2 0 118.11 0; 3 118.11 118.11 0; 4 118.11 0 0; 5 0 236.22 0
9. 6 118.11 236.22 0; 7 0 354.331 0; 8 118.11 354.331 0; 9 0 472.441 0
10. 10 118.11 472.441 0; 11 236.22 118.11 0; 12 236.22 0 0; 13 236.22 236.22 0
11. 14 236.22 354.331 0; 15 236.22 472.441 0
12. MEMBER INCIDENCES
13. 1 1 2; 2 2 3; 3 3 4; 4 2 5; 5 5 6; 6 6 3; 7 5 7; 8 7 8; 9 8 6; 10 7 9; 11 9 10
14. 12 10 8; 13 3 11; 14 6 13; 15 8 14; 16 10 15; 17 11 12; 18 13 11; 19 14 13
15. 20 15 14
16. DEFINE MATERIAL START
17. ISOTROPIC STEEL
18. E 29732.7
19. POISSON 0.3
20. DENSITY 0.000283
21. ALPHA 1.2E-05
22. DAMP 0.03
23. END DEFINE MATERIAL
24. MEMBER PROPERTY INDIAN
```

25. 1 3 4 6 7 9 10 12 17 TO 20 TABLE ST ISHB450
26. 2 5 8 11 13 TO 16 TABLE ST ISHB225
27. CONSTANTS
28. MATERIAL STEEL ALL
29. SUPPORTS
30. 1 4 12 FIXED
31. DEFINE PUSHOVER DATA
32. FRAME 2
33. GNONL 1
34. SAVE LOADSTEP RESULT DISP 0.010000
35. FYE 36.000000 ALL
36. LDSTEP 500
37. SPECTRUM PARAMETERS
38. DAMPING 5.000039. SC 1
40. SS 1
41. S1 1
42. DISP X 5 JOINT 15
43. HINGE FEMA ALL
44. VDB 3
45. END PUSHOVER DATA

***WARNING: BASE SHEAR TO BE DISTRIBUTED IS NOT DEFINED.

10% OF GRAVITY LOAD IS DISTRIBUTED AS LATERAL LOAD.

46. LOAD 1 LOADTYPE GRAVITY
47. SELFWEIGHT Y -1
48. MEMBER LOAD
49. 2 5 8 11 13 TO 16 UNI GY -0.6
50. PERFORM PUSHOVER ANALYSIS

P R O B L E M S T A T I S T I C S

NUMBER OF JOINTS 15 NUMBER OF MEMBERS 20

NUMBER OF PLATES 0 NUMBER OF SOLIDS 0

NUMBER OF SURFACES 0 NUMBER OF SUPPORTS 3

Using 64-bit analysis engine.

SOLVER USED IS THE IN-CORE ADVANCED MATH SOLVER

TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 72

TOTAL LOAD COMBINATION CASES = 0 SO FAR.

*** WARNING:

Pivoting Applied for Small or Zero Pivots. Solution Successfully Completed.

EIGEN METHOD : SUBSPACE

NUMBER OF MODES REQUESTED = 6
NUMBER OF EXISTING MASSES IN THE MODEL = 36
NUMBER OF MODES THAT WILL BE USED = 6
*** EIGENSOLUTION: ADVANCED METHOD ***

3D Concentric Braced Frames

```
****************************************************
* *
* STAAD.Pro CONNECT Edition *
* Version 22.04.00.40 *
* Proprietary Program of *
* Bentley Systems, Inc. *
* Date= JUN 4, 2020 *
* Time= 16:10:46 *
* *
* Licensed to: Shri Vithal Education & Research *
****************************************************
```

1. STAAD SPACE
INPUT FILE: G:\MTech_Pr\Dynamic_Pushover_Trial2-1nobracing.STD
2. START JOB INFORMATION
3. ENGINEER DATE 14-JAN-15
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. SET NL 200
7. UNIT FEET KIP
8. JOINT COORDINATES
9. 1 0 0 0; 2 0 0 20; 3 0 0 40; 4 0 0 60; 5 25 0 0; 6 25 0 20; 7 25 0 40
10. 8 25 0 60; 9 50 0 0; 10 50 0 20; 11 50 0 40; 12 50 0 60; 13 75 0 0; 14 75 0 20
11. 15 75 0 40; 16 75 0 60; 17 100 0 0; 18 100 0 20; 19 100 0 40; 20 100 0 60
12. 21 0 10 0; 22 0 10 20; 23 0 10 40; 24 0 10 60; 25 25 10 0; 26 25 10 20
13. 27 25 10 40; 28 25 10 60; 29 50 10 0; 30 50 10 20; 31 50 10 40; 32 50 10 60
14. 33 75 10 0; 34 75 10 20; 35 75 10 40; 36 75 10 60; 37 100 10 0; 38 100 10 20
15. 39 100 10 40; 40 100 10 60; 50 0 20 0; 51 0 20 20; 52 0 20 40; 53 0 20 60
16. 62 25 20 0; 63 25 20 20; 64 25 20 40; 65 25 20 60; 74 50 20 0; 75 50 20 20
17. 76 50 20 40; 77 50 20 60; 86 75 20 0; 87 75 20 20; 88 75 20 40; 89 75 20 60
18. 98 100 20 0; 99 100 20 20; 100 100 20 40; 101 100 20 60; 104 0 30 0
19. 105 0 30 20; 106 0 30 40; 107 0 30 60; 116 25 30 0; 117 25 30 20; 118 25 30 40
20. 119 25 30 60; 128 50 30 0; 129 50 30 20; 130 50 30 40; 131 50 30 60
21. 140 75 30 0; 141 75 30 20; 142 75 30 40; 143 75 30 60; 152 100 30 0
22. 153 100 30 20; 154 100 30 40; 155 100 30 60; 156 50 40 0; 157 50 40 20
23. 158 50 40 40; 159 50 40 60; 168 75 40 0; 169 75 40 20; 170 75 40 40

24. 171 75 40 60; 180 100 40 0; 181 100 40 20; 182 100 40 40; 183 100 40 60
25. MEMBER INCIDENCES
26. 1 21 1; 2 22 2; 3 23 3; 4 24 4; 5 25 5; 6 26 6; 7 27 7; 8 28 8; 9 29 9
27. 10 30 10; 11 31 11; 12 32 12; 13 33 13; 14 34 14; 15 35 15; 16 36 16; 17 37 17
28. 18 38 18; 19 39 19; 20 40 20; 21 21 22; 22 22 23; 23 23 24; 24 25 21; 25 22 26
29. 26 23 27; 27 24 28; 28 25 26; 29 26 27; 30 27 28; 31 25 29; 32 26 30; 33 27 31
30. 34 28 32; 35 29 30; 36 30 31; 37 31 32; 38 29 33; 39 30 34; 40 31 35; 41 32 36
31. 42 33 34; 43 34 35; 44 35 36; 45 33 37; 46 34 38; 47 35 39; 48 36 40; 49 37 38
32. 50 38 39; 51 39 40; 66 50 21; 67 51 22; 68 52 23; 69 53 24; 70 62 25; 71 63 26
33. 72 64 27; 73 65 28; 74 74 29; 75 75 30; 76 76 31; 77 77 32; 78 86 33; 79 87 34
34. 80 88 35; 81 89 36; 82 98 37; 83 99 38; 84 100 39; 85 101 40; 96 51 50
35. 97 52 51; 98 53 52; 99 50 62; 100 51 63; 101 52 64; 102 53 65; 117 63 62
36. 118 64 63; 119 65 64; 120 62 74; 121 63 75; 122 64 76; 123 65 77; 138 75 74
37. 139 76 75; 140 77 76; 141 74 86; 142 75 87; 143 76 88; 144 77 89; 159 87 86
38. 160 88 87; 161 89 88; 162 86 98; 163 87 99; 164 88 100; 165 89 101; 180 99 98
39. 181 100 99; 182 101 100; 187 104 50; 188 105 51; 189 106 52; 190 107 53
40. 191 116 62; 192 117 63; 193 118 64; 194 119 65; 195 128 74; 196 129 75
41. 197 130 76; 198 131 77; 199 140 86; 200 141 87; 201 142 88; 202 143 89
42. 203 152 98; 204 153 99; 205 154 100; 206 155 101; 211 105 104; 212 106 105
43. 213 107 106; 214 104 116; 215 105 117; 216 106 118; 217 107 119; 232 117 116
44. 233 118 117; 234 119 118; 235 116 128; 236 117 129; 237 118 130; 238 119 131
45. 253 129 128; 254 130 129; 255 131 130; 256 128 140; 257 129 141; 258 130 142
46. 259 131 143; 274 141 140; 275 142 141; 276 143 142; 277 140 152; 278 141 153
47. 279 142 154; 280 143 155; 295 153 152; 296 154 153; 297 155 154; 298 156 128
48. 299 157 129; 300 158 130; 301 159 131; 302 168 140; 303 169 141; 304 170 142
49. 305 171 143; 306 180 152; 307 181 153; 308 182 154; 309 183 155; 310 157 156
50. 311 158 157; 312 159 158; 313 156 168; 314 157 169; 315 158 170; 316 159 171
51. 331 169 168; 332 170 169; 333 171 170; 334 168 180; 335 169 181; 336 170 182
52. 337 171 183; 352 181 180; 353 182 181; 354 183 182
53. DEFINE MATERIAL START
54. ISOTROPIC STEEL
55. E 4.176E+06
56. POISSON 0.3
57. DENSITY 0.489024
58. ALPHA 6.5E-06
59. DAMP 0.03
60. TYPE STEEL
61. STRENGTH RY 1.5 RT 1.2
62. END DEFINE MATERIAL
63. MEMBER PROPERTY INDIAN

64. 2 3 6 7 14 15 67 68 71 72 79 80 188 189 192 193 200 201 303 -
65. 304 TABLE ST ISLB450
66. 25 26 32 33 39 40 46 47 TABLE ST ISLB500
67. 21 TO 24 27 TO 31 34 TO 38 41 TO 45 48 TO 51 96 TO 102 117 TO 123 138 TO 144 -
68. 159 TO 165 180 TO 182 211 TO 217 232 TO 238 253 TO 259 274 TO 280 -
69. 295 TO 297 310 TO 316 331 TO 337 352 TO 354 TABLE ST ISHB200
70. 1 4 5 8 10 11 13 16 TO 20 66 69 70 73 TO 78 81 TO 85 187 190 191 194 TO 199 -
71. 202 TO 206 298 TO 302 305 TO 309 TABLE ST ISHB350H
72. 9 12 TABLE ST ISLB550
73. CONSTANTS
74. MATERIAL STEEL ALL
75. SUPPORTS
76. 1 TO 20 FIXED
77. UNIT METER KN
78. DEFINE PUSHOVER DATA
79. FRAME 2
80. GNONL 0
81. LDSTEP 250
82. SPECTRUM PARAMETERS
83. DAMPING 5.0000
84. SC 4
85. SS 1
86. S1 1
87. DISP X 1.2 JOINT 158
88. END PUSHOVER DATA

***WARNING: BASE SHEAR TO BE DISTRIBUTED IS NOT DEFINED.

10% OF GRAVITY LOAD IS DISTRIBUTED AS LATERAL LOAD.

89. UNIT FEET KIP
90. LOAD 1 LOADTYPE GRAVITY TITLE LOAD CASE 1
91. SELFWEIGHT Y -1
92. UNIT METER KN
93. MEMBER LOAD
94. 21 TO 51 96 TO 102 117 TO 123 138 TO 144 159 TO 165 180 TO 182 211 TO 217 -
95. 232 TO 238 253 TO 259 274 TO 280 295 TO 297 310 TO 316 331 TO 337 -
96. 352 TO 354 UNI GY -3
97. UNIT FEET KIP
98. PERFORM PUSHOVER ANALYSIS

P R O B L E M S T A T I S T I C S

NUMBER OF JOINTS 92 NUMBER OF MEMBERS 182

NUMBER OF PLATES 0 NUMBER OF SOLIDS 0
NUMBER OF SURFACES 0 NUMBER OF SUPPORTS 20
Using 64-bit analysis engine.
SOLVER USED IS THE IN-CORE ADVANCED MATH SOLVER
TOTAL PRIMARY LOAD CASES = 1, TOTAL DEGREES OF FREEDOM = 432
TOTAL LOAD COMBINATION CASES = 0 SO FAR.
EIGEN METHOD : SUBSPACE

NUMBER OF MODES REQUESTED = 6
NUMBER OF EXISTING MASSES IN THE MODEL = 216
NUMBER OF MODES THAT WILL BE USED = 6
*** EIGENSOLUTION: ADVANCED METHOD ***
CALCULATED FREQUENCIES FOR LOAD CASE 2
MODE FREQUENCY(CYCLES/SEC) PERIOD(SEC)
1 0.090 11.07621
2 0.109 9.14968
3 0.569 1.75597
4 0.674 1.48366
5 0.783 1.27775
6 0.984 1.01669
MODAL WEIGHT (MODAL MASS TIMES g) IN KIP GENERALIZED
MODE X Y Z WEIGHT
1 0.000000E+00 0.000000E+00 4.004865E+02 2.189960E+02
2 0.000000E+00 0.000000E+00 1.033348E-11 1.108830E+02
3 0.000000E+00 0.000000E+00 1.196171E+02 2.445621E+02
4 0.000000E+00 0.000000E+00 2.713379E-12 1.506619E+02
5 4.519506E+02 3.022909E-16 5.910363E-47 2.644991E+02
6 2.889758E-34 6.598964E-51 7.152078E-03 2.250205E+02
MASS PARTICIPATION FACTORS
MASS PARTICIPATION FACTORS IN PERCENT

MODE X Y Z SUMM-X SUMM-Y SUMM-Z
1 0.00 0.00 69.62 0.000 0.000 69.621
2 0.00 0.00 0.00 0.000 0.000 69.621
3 0.00 0.00 20.79 0.000 0.000 90.415
4 0.00 0.00 0.00 0.000 0.000 90.415
5 78.57 0.00 0.00 78.567 0.000 90.415
6 0.00 0.00 0.00 78.567 0.000 90.416
***WARNING: MEMBER # 3 HAS FAILED IN " FORCE-CONTROLLED ACTION
WHERE INTERACTION RATIO EXCEEDS 1.0.

**WARNING : THE DISPLACEMENT AT JOINT 15 EXCEEDS THE SPECIFIED DISPLACEMENT.

51. FINISH

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= MAY 21,2020 TIME= 15:47: 8 ****

****

* For technical assistance on STAAD.Pro, please visit *

* http://www.bentley.com/en/support/ *

* *

* Details about additional assistance from *

* Bentley and Partners can be found at program menu *

* Help->Technical Support *

* *

* http://www.bentley.com *

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