Performance Evaluation of 2-D Moment Frames with High Strength Steel End-Plate Connections

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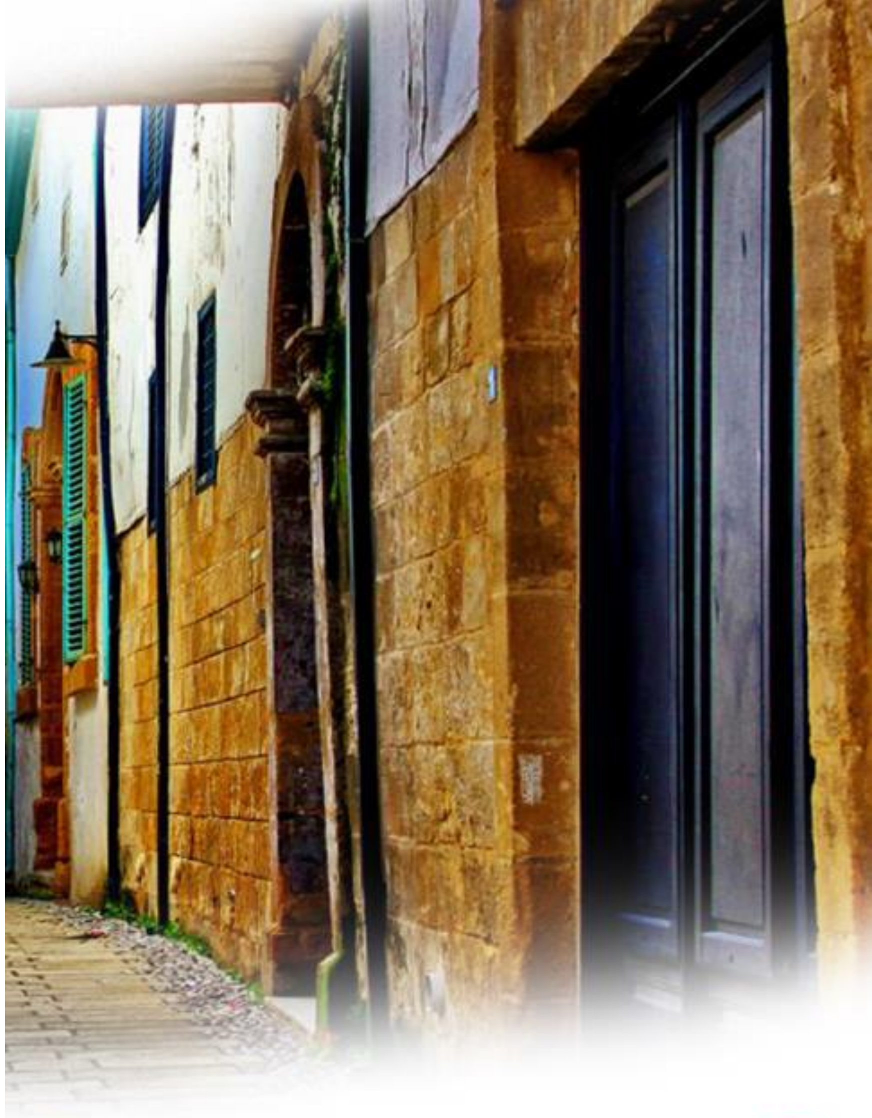


Why High Strength Steels (HSS)?

- Reduction in *steel consumption*, overall weight of the structure,
- Hence reduce the construction impact on environment and contribute to the *sustainability* of natural resources
- *Savings:* fabrication, transportation to site and erection.
- HSS sections are lighter, more slender, *more desirable for architecture*, allow for more space and freedom within the Buildings
- HSS exhibit **high yield ratios** and **limited deformation capacity**
- Mechanical properties: *Higher performance* in tensile stress, toughness, weldability, cold forming and corrosion resistance
- HSS **may cause a change** in the structural seismic performance

Introduction

Why increase in demand for HSS?

- Increase in the demand for constructing **tall structures**
- Material and cost saving, **sustainability**
- Problems with steel beam-to-column connections during **Northridge** and **Kobe** earthquakes

Researchers encouraged to embark on research into **moment connections** employing

HSS columns, beams, end-plates and bolts.

What are the adverse effects of HSS compared to mild steel?

- Higher **elastic strength** but may not have **higher modulus of elasticity**.

  If design is *governed by stiffness*, the *serviceability limit state* criteria may not be achieved
- **Significant differences** in terms of
  - ✓ stress-strain curves
  - ✓ residual stress distributions
  - ✓ different effects of initial imperfections
- These would lead to significant differences in **local and overall buckling behaviour** of structures which may cause **frame stability problems.**

What are the adverse effects of HSS?

- There is *limited study on **HSS** frame behaviour.*
- Most of the work done is about *HSS connections* and *column members*.
- There is need for understanding the behaviour of
  - ✓ **HSS** frame and **HSS** beam to column joints

particularly when frames are subjected to abnormal loading, such as,

earthquake, **fire** or **blast** loading

What are the adverse effects of HSS?

- Beam to column joints must be designed for
  - ✓ strength
  - ✓ stiffness
  - ✓ rotation capacity
  - ✓ ductility
- The full non-linear moment-rotation responses of joints are required for the **global frame analysis**.

Objective of study

Investigating the performance of 2-D moment frames in terms of

strength

stiffness

ductility

by using **HSS** and **mild steel** end-plate beam to column connections

Methodology

A typical 2-D moment frame from an existing steel framed residential building is used as a case study so that the behaviour of frames can be compared.

1) **FEM** was used to **validate the experimental results** available from past research on HSS end-plate connections.

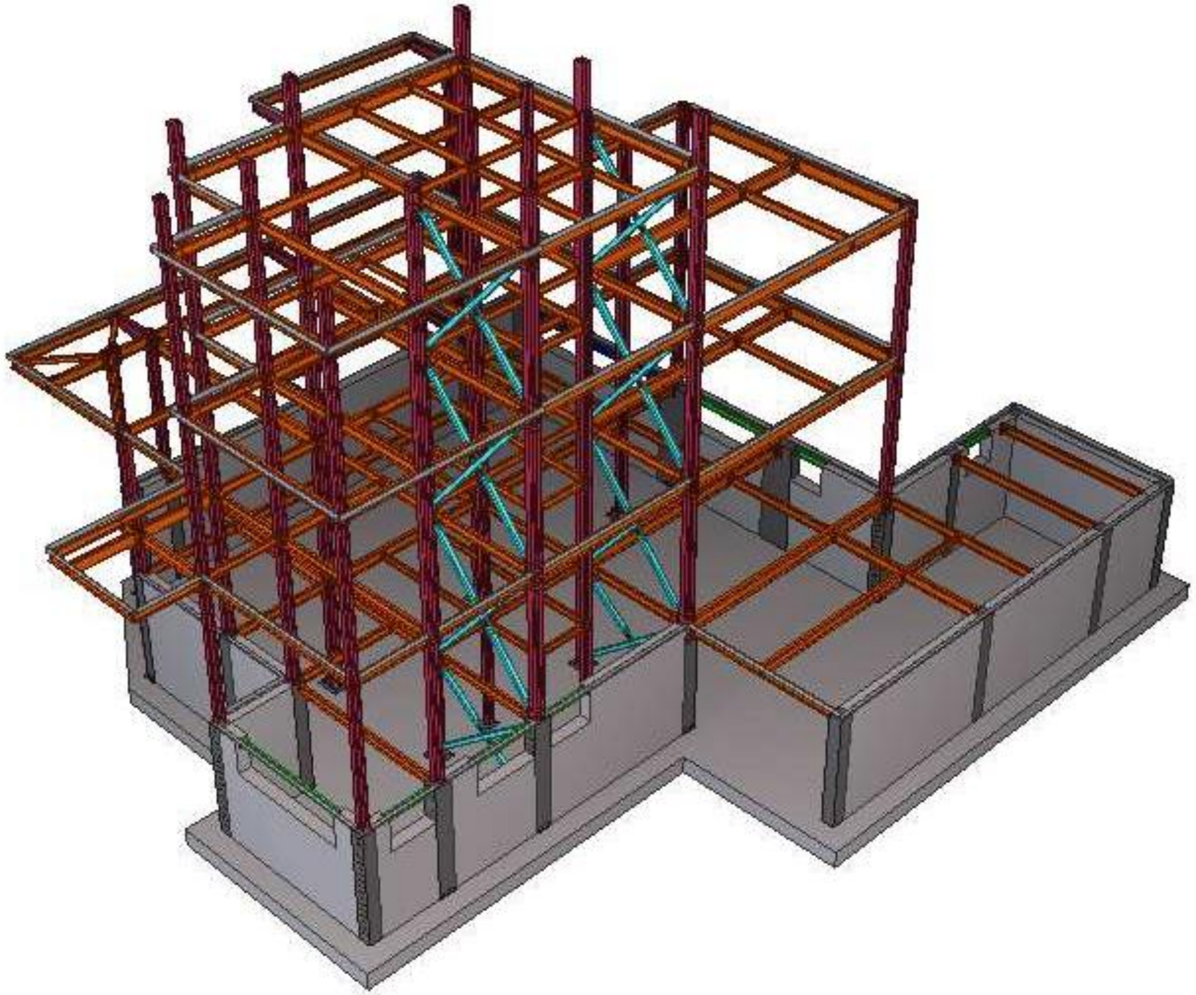
2) The **moment curvature relationship** was captured for the purpose of finding hinge properties.

3) **Time-history analysis** (dynamic analysis) was used with real earthquake data (*Duzce Earthquake-magnitude 7.14*) in SAP2000 software to obtain the **local and global performance**.

4) This performance was then compared with the **performance levels given by FEMA 356.**

Case Study

Case Study



Case Study

Case Study

- Residential house, 550 m^2 floor area + 55 m^2 Depo + 50 m^2 Swimming Pool Area
- Location: **Iskele-Bogaz, North Cyprus**
- Grade **S275** steel frame, except cross bracing, **S235** hollow section
- **Moment frame** in one direction and **braced frame** in the other direction.
- Turkish Earthquake Code,

  Earthquake **Zone 2**

  Design **ground acceleration 0.3**

  Importance factor **1. 0**

  Behaviour factor **5.0** for braced frame

  8.0 for moment frame

  The allowable bearing capacity of soil is 180 kN/m^2
- **IPE** beams and **HEB** columns
- Beams are connected to column flanges via

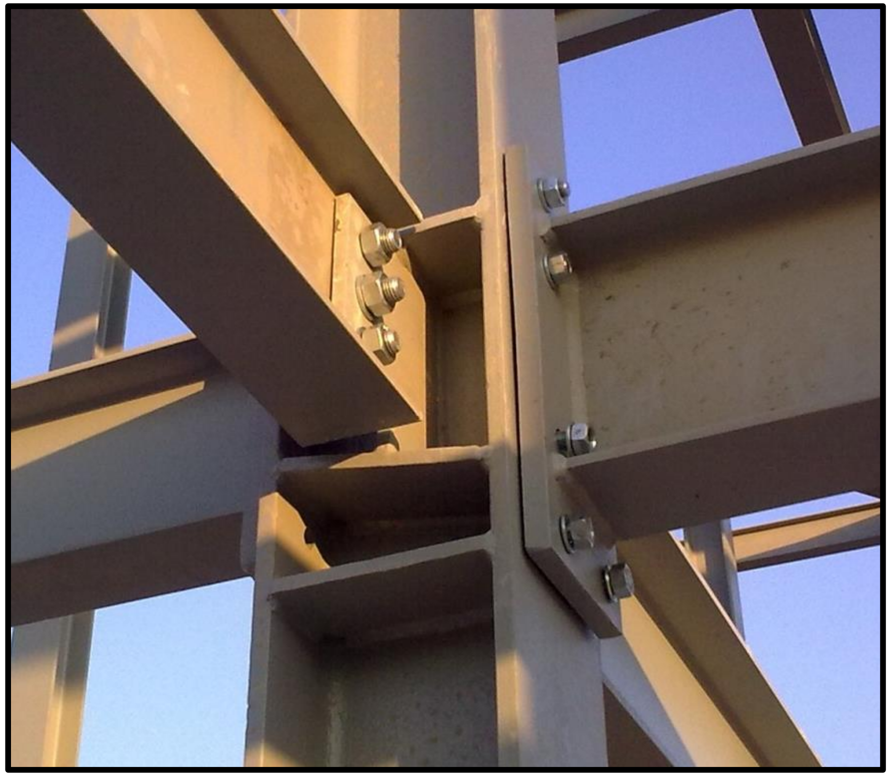
  20 mm thick grade **S275** extended end-plate connections

  8M16, grade **10.9** bolts.

Case Study

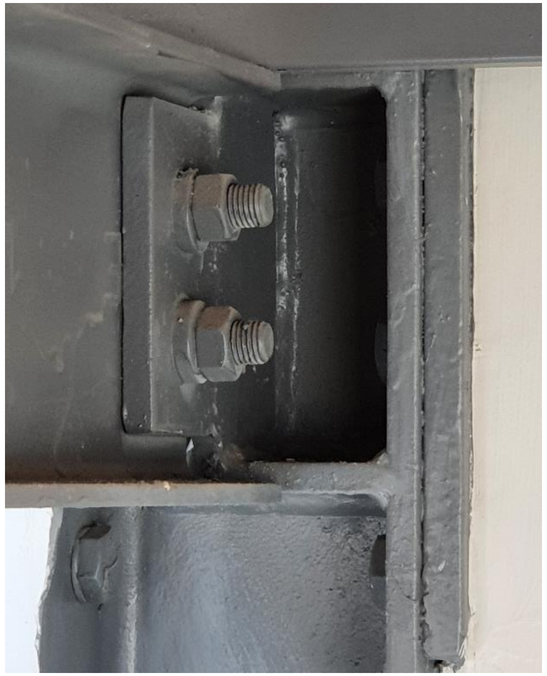
Case Study

Simple and **Moment Connections:**
Fin Plate and **Extended End Plate**



Case Study

Simple Connections:
Fin Plate



Moment Connections:
Extended End Plate

ABAQUS - Modeling of connections

5 different hinges to be formed in the selected frame.

H1 **H1R** **H2** **H4** **H4L**

H **:** hinge

1,2,4 : bays starting from axis E

L, R **:** hinge on the left, right side of the beam

GROUPS: **G1:** hinges with 20 mm S275 plate

G2: hinges with 8 mm S275 plate

G3: hinges with 8 mm S690 (HSS) plate

Beam section, column section and end plate geometrical dimensions are the same for each group

ABAQUS - Modeling of connections

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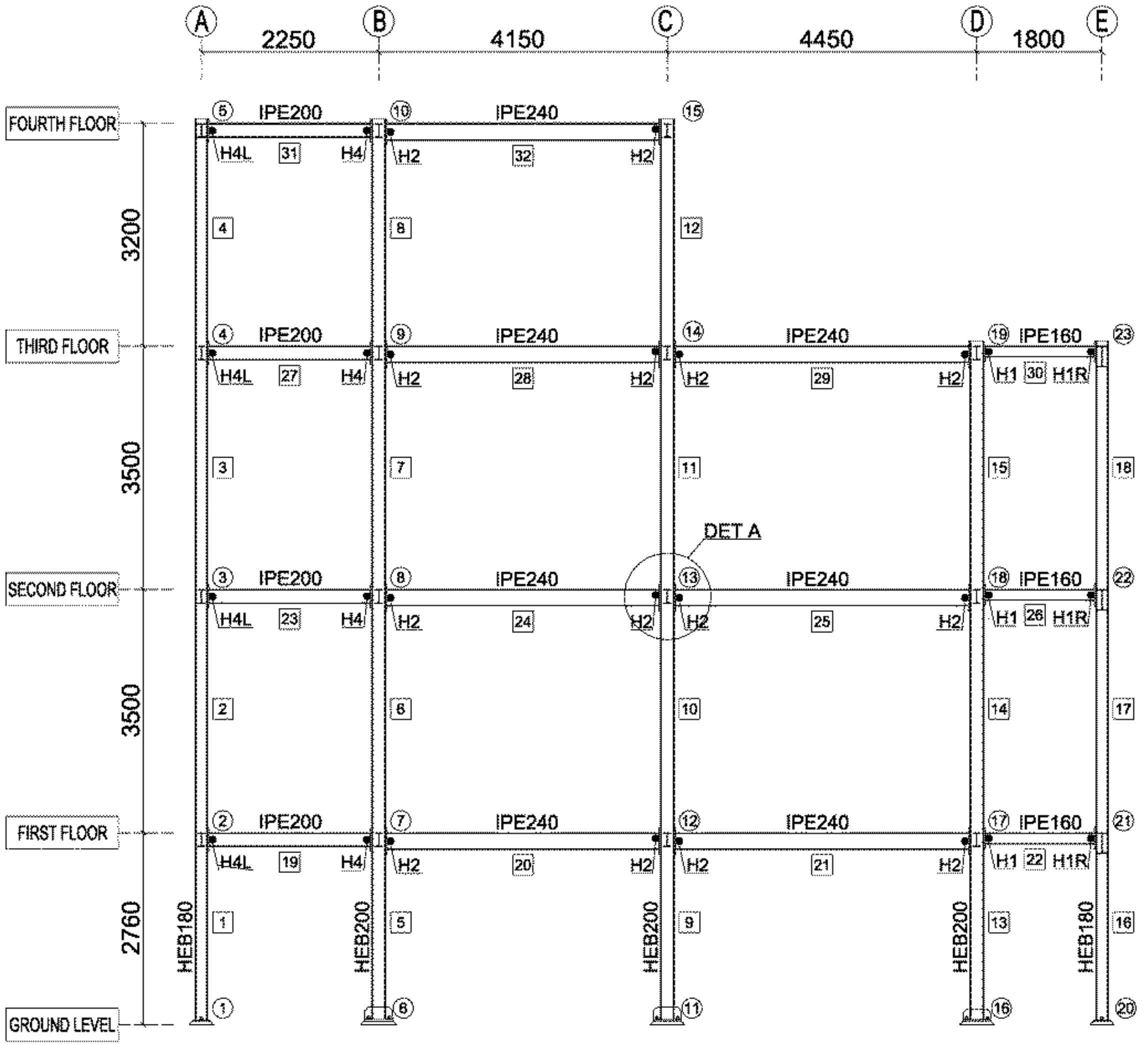
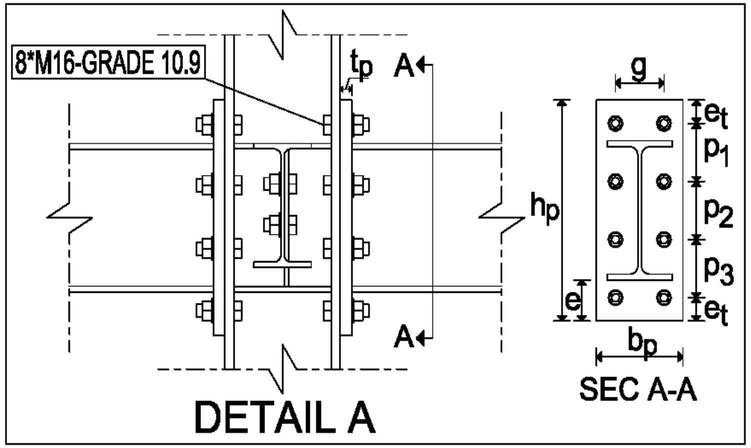


ABAQUS - Modeling of connections

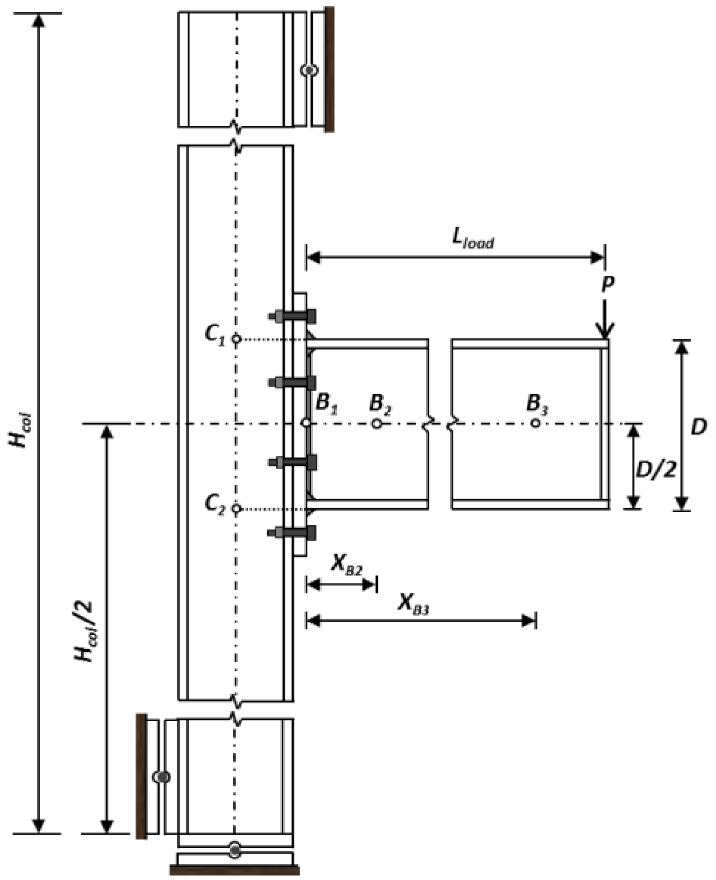


G1 Specimen (tp=20mm) (S275)	G2 Specimen (tp=8mm) (S275)	G3 Specimen (tp=8mm) (S690)	Column Sections (S275)	Beam Sections (S275)	End-plate							
					h_P	b_p	g	e_t	e	p_1	p_2	p_3
H1-20	H1-8	HSS-H1-8	HE200B	IPE160	295	160	90	30	67	85	65	85
HR1-20	HR1-8	HSS-HR1-8	HE180B	IPE160	295	160	90	30	67	85	65	85
H2-20	H2-8	HSS-H2-8	HE200B	IPE240	380	160	90	30	70	80	160	80
H4-20	H4-8	HSS-H4-8	HE200B	IPE200	380	160	90	30	70	80	160	80
H4L-20	H4L-8	HSS-H4L-8	HE180B	IPE200	380	160	90	30	70	80	160	80

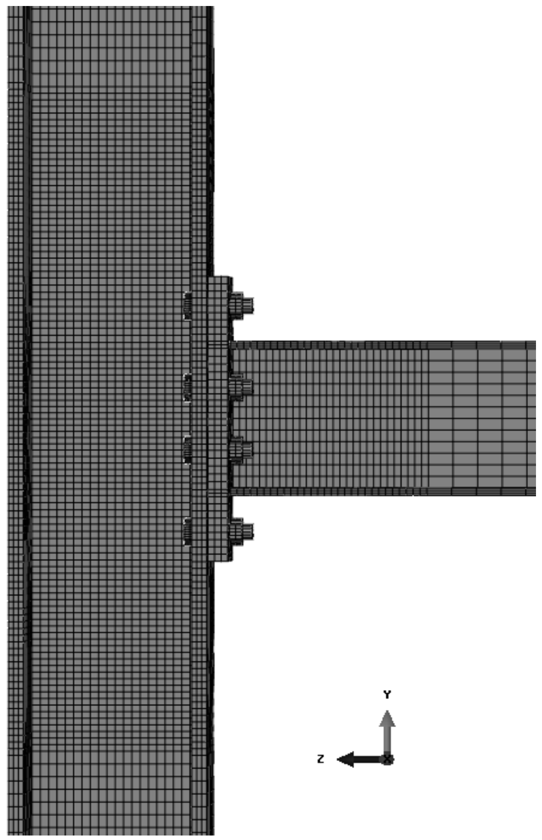
ABAQUS - Modeling of connections

- ABAQUS/ Standard: 3-D nonlinear FE simulations
  - ✓ General-purpose FE explicit solver
  - ✓ Length of column, H_{col} = 3625 mm
  - ✓ Length of beam = 1550 mm
  - ✓ Stiffener thickness at beam loading point is equal to beam flange thickness
  - ✓ Half of the structure was modelled, due to symmetry
  - ✓ Bottom of the column pinned in three directions
  - ✓ Top of the column pinned in two directions, movement allowed along the column axis
- **Quadri-linear** stress–strain curve used to define the *materials properties* for the joint
- The values of the *yield and ultimate stresses* obtained from **EC3**
- *Material and geometric non-linearities* were considered to capture large deformation and local instability effects
- The **solid element C3D8R** was used to model the **HSS connection** components.

ABAQUS - Modeling of connections



(a)



(b)

The generation of the **FE mesh** is controlled by three parameters.

- No. of elements through the thickness, $n_{et} = 3$
- Length of the elements in the region

  near the connection, $l_{en} = 7$ mm

  far from the connection, $l_{ef} = 25$ mm

Extended end-plate connection ***flexural behaviour***

Moment-rotation (*M-ø*) curve.

Bending moment = $P \times L_{load}$

Rotational deformation

= connection rotational deformation, ϕ_c + shear deformation of the column web panel zone, γ

Rotational deformation of joint is based on

the *vertical displacement measurements of the reference points*

B1 to B3, C1 and C2 (Díaz et al. and Coelho and Bijlaard)

Verification of FEM Simulations

Three HSS end-plate connections tested by **Coelho and Bijlaard** were used

End-plate thicknesses: 10.1 mm, 14.62 mm

Bolt grades: 12.9, 8.8 6M24 - hand tightened without pre-loads

Mechanical properties of end-plates were obtained from tensile coupon tests in ref.

Table 2: Details of test specimens

Test	Column Section (S355J2)	Beam Section (S355J2)	End plate (S690)								
			h_P	b_p	t_p	g	e_t	e_c	e	p_1	p_2
EEP_10_2a*	HE300M	HE320A	435	300	10.10	150	40	100	25	160	135
EEP_10_2b¤	HE300M	HE320A	435	300	10.10	150	40	100	25	160	135
EEP_15_a*	HE300M	HE320A	435	300	14.62	150	40	100	25	160	135

* Grade 12.9 was used for 6 M24 standard bolts

¤ Grade 8.8 was used for 6 M24 standard bolts

Comparison Tests - FEA

The results of **tests** and **FE** models are in very good agreement.

Initial stiffness of the joint, $S_{j,in,}$

Ultimate flexural resistance of joint, $M_{j,max}$

Rotation corresponding to ultimate flexural resistance, $\phi_{j,max}$

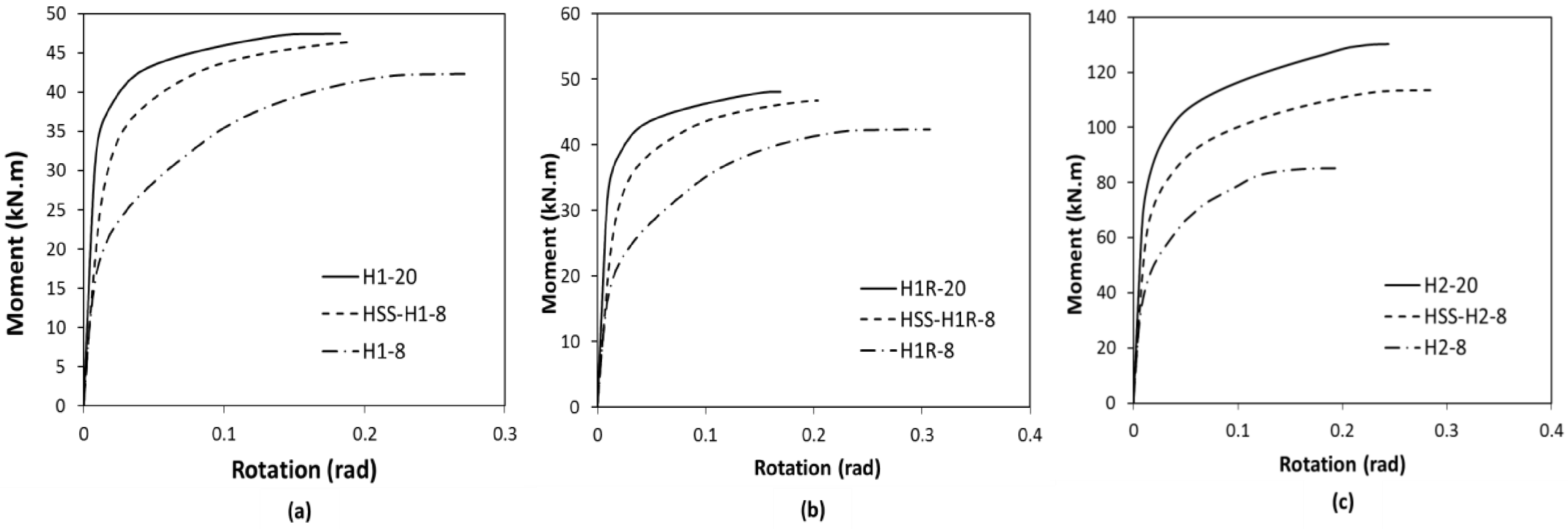
Test	Test [13]			FE					
	$S_{j,in,Exp}$ (kN.m/rad)	$M_{j,max,Exp}$ (kN.m)	$\phi_{j,max,Exp}$ (rad)	$S_{j,in,FE}$ (kN.m/rad)	$M_{j,max,FE}$ (kN.m)	$\phi_{j,max,FE}$ (rad)	$\frac{S_{j,in,Exp}}{S_{j,in,FE}}$	$\frac{M_{j,max,Exp}}{M_{j,max,FE}}$	$\frac{\phi_{j,max,Exp}}{\phi_{j,max,FE}}$
EEP_10_2a	17200	244.00	0.0360	17298	239.36	0.0373	0.994	1.019	0.964
EEP_10_2b	17308	252.00	0.0370	19900	233.66	0.0405	0.870	1.079	0.913
EEP_15_a	35300	366.00	0.0200	32684	367.17	0.0219	1.080	0.997	0.912
Mean	-	-	-	-	-	-	0.981	1.032	0.930
COV	-	-	-	-	-	-	0.108	0.041	0.032

M-ϕ Comparison for Same Hinges -

Three Different Groups

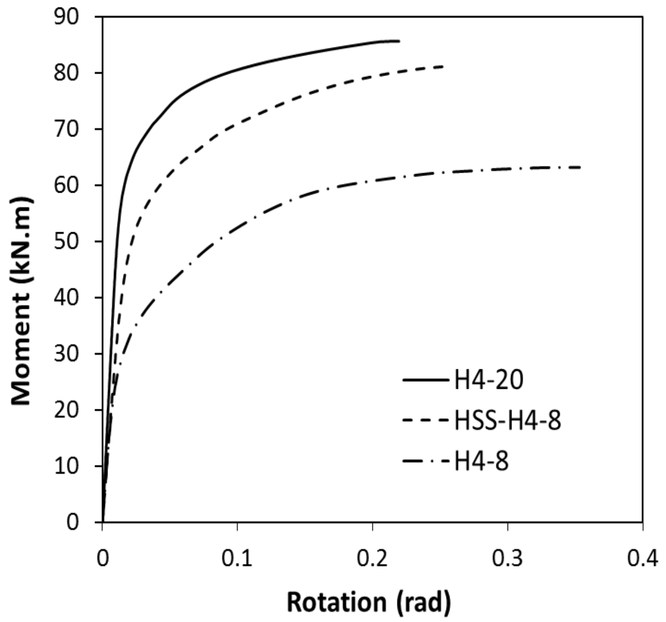
M-ø Comparison for Same Hinges - Three Different Groups

- Group 1 has the *highest strength* and *initial stiffness*
- Hinges with **20 mm** end plate has the lowest rotational capacity.
- Hinges with **8mm** (except H2) has the best rotational capacity

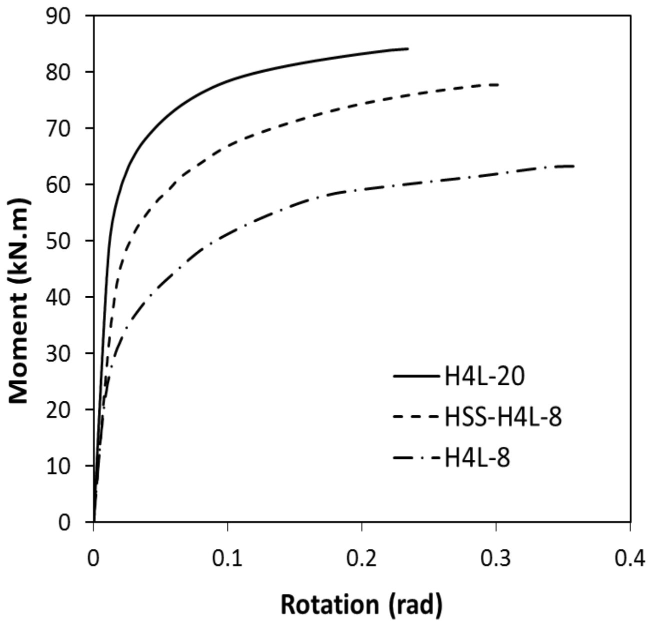


M-ø Comparison for Same Hinges - Three Different Groups

- Group 1 has the *highest strength* and *initial stiffness*
- Hinges with **20 mm** end plate has the lowest rotational capacity.
- Hinges with **8mm** (except H2) has the best rotational capacity
- H4L-20 has 26%, 39% more strength and initial stiffness than H4L-8, respectively



(d)



(e)

FE Results - Main characteristics of idealized moment-rotation curves

- The **initial and post yield stiffness** values are computed by means of regression analysis of the elasto-plastic branches before and after the knee range.
- Plastic flexural resistance $M_{j,R,FE}$
- Maximum bending moment $M_{j,max,FE}$ and corresponding rotation values

The idealized (elasto-plastic) moment curvature relationships are captured for the purpose of finding hinge properties.

Group	Specimen	Resistance (kN.m)		Rotation (rad)		Failure mode
		$M_{j,R,FE}$	$M_{j,max,FE}$	ϕ_y	$\phi_{j,max,FE}$	
G1	H1-20	42.9	48.3	0.0106	0.1863	BF
	HR1-20	43.1	48.9	0.0116	0.1810	BF
	H2-20	106.7	133.2	0.0132	0.2480	BF
	H4-20	74.8	87.4	0.0155	0.2266	BF
	H4L-20	75.1	85.9	0.0165	0.2603	BF
G2	H1-8	35.1	43.1	0.0153	0.2708	EF
	HR1-8	34.5	43.1	0.0169	0.2840	EF
	H2-8	77.8	85.2	0.0143	0.1991	EF
	H4-8	57.2	63.3	0.0187	0.3302	EF
	H4L-8	51.8	63.5	0.0186	0.3419	EF
G3	HSS-H1-8	40.5	47.3	0.0177	0.2005	BF
	HSS-HR1-8	40.5	47.9	0.0194	0.2181	BF
	HSS-H2-8	97.2	113.7	0.0176	0.2646	EF/BOF
	HSS-H4-8	65.3	84.5	0.0211	0.2711	BF
	HSS-H4L-8	64.7	78.8	0.0231	0.3004	EF/BOF

Note: BF denotes Beam Faliure , EF denotes End-plate Faliure, BOF denotes Bolt Faliure.

Time-History *Analysis*

Earthquake Hazard Levels (probability of exceedance, 50 years)

- **Federal Emergency Management Agency (FEMA 356)**

  50%, 20%, 10% and 2%

- **Turkish Earthquake Code (TEC)**

  50%, 10% and 2%

Performance level definitions and evaluation

According to FEMA 356, there are four main structural performance levels;

Operational Level (OP)

Immediate Occupancy (IO)

Life Safety (LS)

Collapse Prevention (CP)

Structural performance will be assessed at **global** and **member level** by considering the **drift** and **plastic rotation**, respectively.

Permanent and transient drift limit values (FEMA 356) for steel frames are used.

Column and beam ends take most of the stresses during the earthquake excitation.

Target building performance levels

- TEC specifies minimum performance levels for buildings according to their usage purposes.
- The case study building is designed for **residential purposes.**
- Expected performance level - minimum of **Life safety** in case of *design earthquake*.

Case study analysis: **TEC** with **10%** probability of exceedance in **50 years** for design earthquake.

Member level evaluation

- The maximum **plastic rotations** from dynamic analysis for members are given in Tables 5-6, depending on the *formation sequence of the plastic hinges*.
- The maximum plastic rotations are compared with the limitations given in FEMA 356 *in terms of performance level.*

The **beam evaluations** of three group models show that the most severe damage is with

G1: 21 members - **IO level**

4 members - **LS level**

1 member - **CP level**

G3: had the best performance and no member reached LS level.

The column evaluations: performance of G2 is the best among all groups.

G2: 3 members - **IO level**

3 members - **LS level**

G1,G3: 3 members - **CP level**

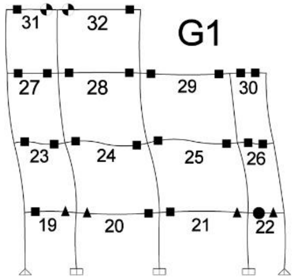
Similar performance

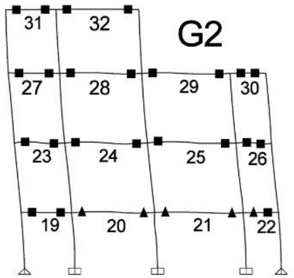
Results and Discussions

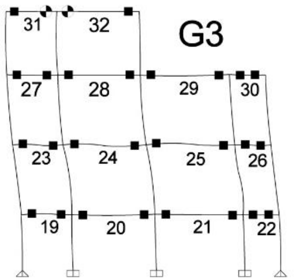
Results and Discussions

Table 5: Plastic formations and performance levels for beams of three groups

		GROUP 1 (G1)					GROUP 2 (G2)					GROUP 3 (G3)				
		Moments	Rotation	FEMA 356 limits (rad)			Moments	Rotation	FEMA 356 limits (rad)			Moments	Rotation	FEMA 356 limits (rad)		
Mem.	Joint	(kNm)	(rad)	IO	LS	CP	(kNm)	(rad)	IO	LS	CP	(kNm)	(rad)	IO	LS	CP
19	2	76.22	*0.0279*	**0.0041**	0.0329	0.0494	52.88	*0.0326*	**0.0046**	0.0371	0.0557	65.99	*0.0287*	**0.0058**	0.0461	0.0692
19	7	76.49	*0.0318*	0.0039	**0.0310**	0.0466	57.82	*0.0326*	**0.0047**	0.0373	0.0560	67.57	*0.0328*	**0.0053**	0.0422	0.0633
20	7	110.01	*0.0307*	0.0033	**0.0264**	0.0396	78.94	*0.0327*	0.0036	**0.0286**	0.0428	99.19	*0.0319*	**0.0044**	0.0352	0.0528
20	12	109.39	*0.0250*	**0.0033**	0.0264	0.0396	78.96	*0.0330*	0.0036	**0.0286**	0.0428	99.05	*0.0297*	**0.0044**	0.0352	0.0528
21	12	109.32	*0.0244*	**0.0033**	0.0264	0.0396	78.94	*0.0326*	0.0036	**0.0286**	0.0428	99.02	*0.0291*	**0.0044**	0.0352	0.0528
21	17	110.02	*0.0309*	0.0033	**0.0264**	0.0396	78.95	*0.0327*	0.0036	**0.0286**	0.0428	99.17	*0.0316*	**0.0044**	0.0352	0.0528
22	17	43.86	*0.0333*	0.0027	0.0212	**0.0319**	36.05	*0.0334*	0.0038	**0.0306**	0.0460	41.57	*0.0333*	**0.0044**	0.0353	0.0530
22	21	44.05	*0.0297*	0.0029	**0.0231**	0.0347	35.46	*0.0330*	**0.0042**	0.0338	0.0507	41.53	*0.0307*	**0.0049**	0.0389	0.0583
23	3	75.95	*0.0212*	**0.0041**	0.0329	0.0494	52.79	*0.0299*	**0.0046**	0.0371	0.0557	65.84	*0.0253*	**0.0058**	0.0461	0.0692
23	8	75.72	*0.0174*	**0.0039**	0.0310	0.0466	57.67	*0.0251*	**0.0047**	0.0373	0.0560	66.76	*0.0209*	**0.0053**	0.0422	0.0633
24	8	108.48	*0.0165*	**0.0033**	0.0264	0.0396	78.68	*0.0252*	**0.0036**	0.0286	0.0428	98.45	*0.0201*	**0.0044**	0.0352	0.0528
24	13	108.43	*0.0161*	**0.0033**	0.0264	0.0396	78.66	*0.0245*	**0.0036**	0.0286	0.0428	98.43	*0.0198*	**0.0044**	0.0352	0.0528
25	13	108.37	*0.0155*	**0.0033**	0.0264	0.0396	78.64	*0.0241*	**0.0036**	0.0286	0.0428	98.40	*0.0193*	**0.0044**	0.0352	0.0528
25	18	108.37	*0.0173*	**0.0033**	0.0264	0.0396	78.70	*0.0256*	**0.0036**	0.0286	0.0428	98.49	*0.0206*	**0.0044**	0.0352	0.0528
26	18	43.46	*0.0198*	**0.0027**	0.0212	0.0319	35.84	*0.0262*	**0.0038**	0.0306	0.0460	41.21	*0.0223*	**0.0044**	0.0353	0.0530
26	22	43.84	*0.0230*	**0.0029**	0.0231	0.0347	35.35	*0.0292*	**0.0042**	0.0338	0.0507	41.36	*0.0258*	**0.0049**	0.0389	0.0583
27	4	75.61	*0.0127*	**0.0041**	0.0329	0.0494	52.38	*0.0175*	**0.0046**	0.0371	0.0557	65.33	*0.0139*	**0.0058**	0.0461	0.0692
27	9	75.38	*0.0109*	**0.0039**	0.0310	0.0466	57.49	*0.0154*	**0.0047**	0.0373	0.0560	66.16	*0.0123*	**0.0053**	0.0422	0.0633
28	9	107.80	*0.0102*	**0.0033**	0.0264	0.0396	78.35	*0.0158*	**0.0036**	0.0286	0.0428	97.91	*0.0114*	**0.0044**	0.0352	0.0528
28	14	107.51	*0.0076*	**0.0033**	0.0264	0.0396	78.35	*0.0140*	**0.0036**	0.0286	0.0428	97.76	*0.0090*	**0.0044**	0.0352	0.0528
29	14	107.45	*0.0070*	**0.0033**	0.0264	0.0396	78.28	*0.0137*	**0.0036**	0.0286	0.0428	97.73	*0.0085*	**0.0044**	0.0352	0.0528
29	19	107.32	*0.0058*	**0.0033**	0.0264	0.0396	78.35	*0.0157*	**0.0036**	0.0286	0.0428	97.82	*0.0100*	**0.0044**	0.0352	0.0528
30	19	43.12	*0.0078*	**0.0027**	0.0212	0.0319	35.57	*0.0165*	**0.0038**	0.0306	0.0460	40.87	*0.0116*	**0.0044**	0.0353	0.0530
30	23	43.55	*0.0143*	**0.0029**	0.0231	0.0347	35.09	*0.0200*	**0.0042**	0.0338	0.0507	41.03	*0.0016*	**0.0049**	0.0389	0.0583
31	5	75.31	*0.0052*	**0.0041**	0.0329	0.0494	51.19	*0.0119*	**0.0046**	0.0371	0.0557	65.07	*0.0082*	**0.0058**	0.0461	0.0692
31	10	74.85	*0.0009*	0.0039	0.0310	0.0466	57.32	*0.0066*	**0.0047**	0.0373	0.0560	65.56	*0.0038*	0.0053	0.0422	0.0633
32	10	106.73	*0.0002*	0.0033	0.0264	0.0396	78.03	*0.0067*	**0.0036**	0.0286	0.0428	97.37	*0.0028*	0.0044	0.0352	0.0528
32	15	107.43	*0.0068*	**0.0033**	0.0264	0.0396	78.21	*0.0117*	**0.0036**	0.0286	0.0428	97.74	*0.0087*	**0.0044**	0.0352	0.0528







Plastic formations and performance levels for beams of three groups

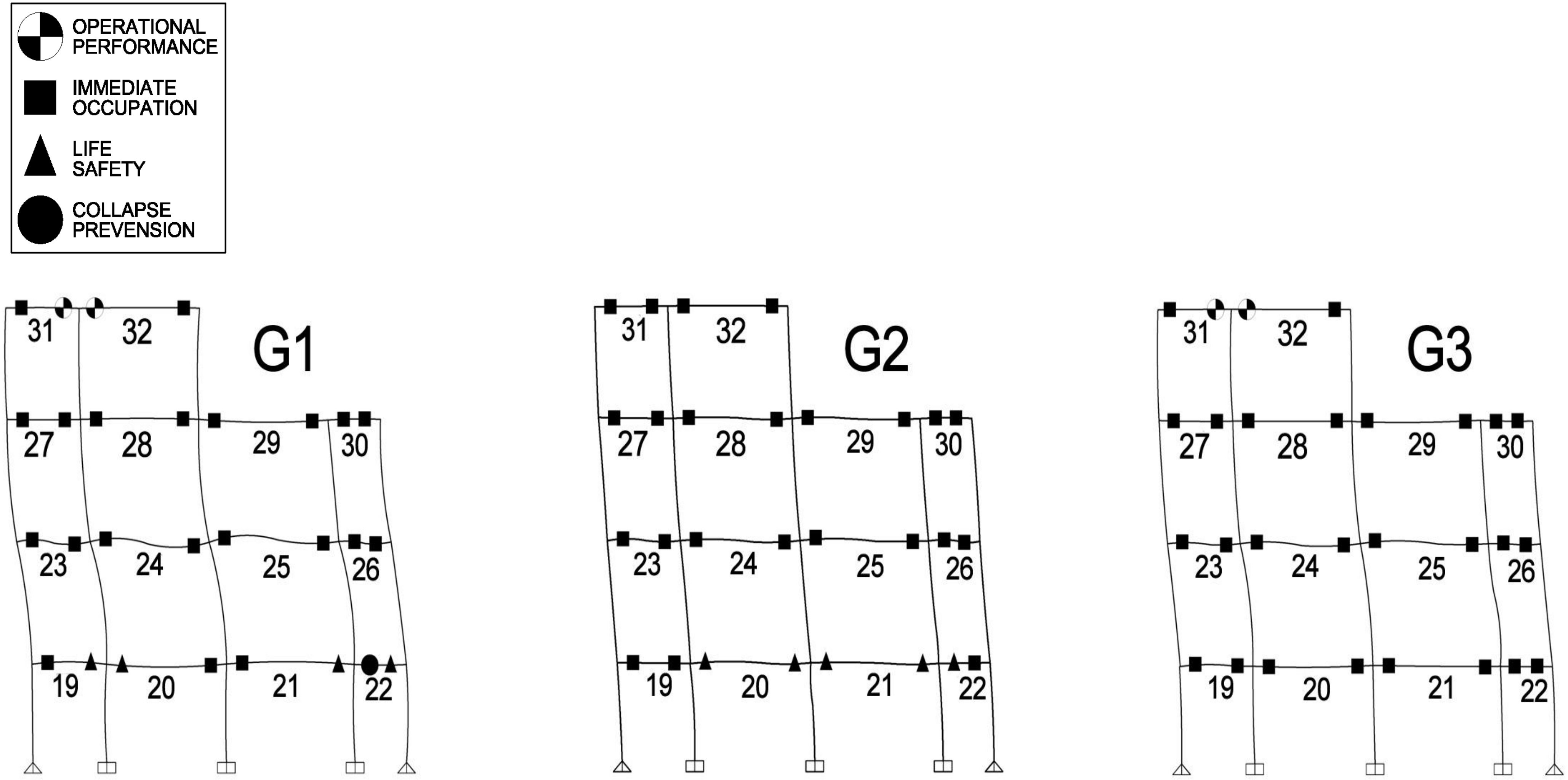
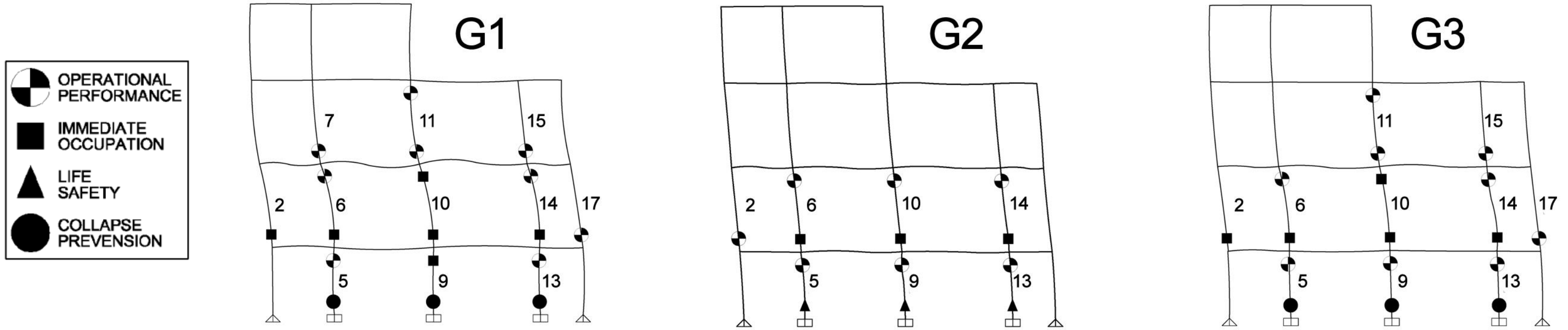
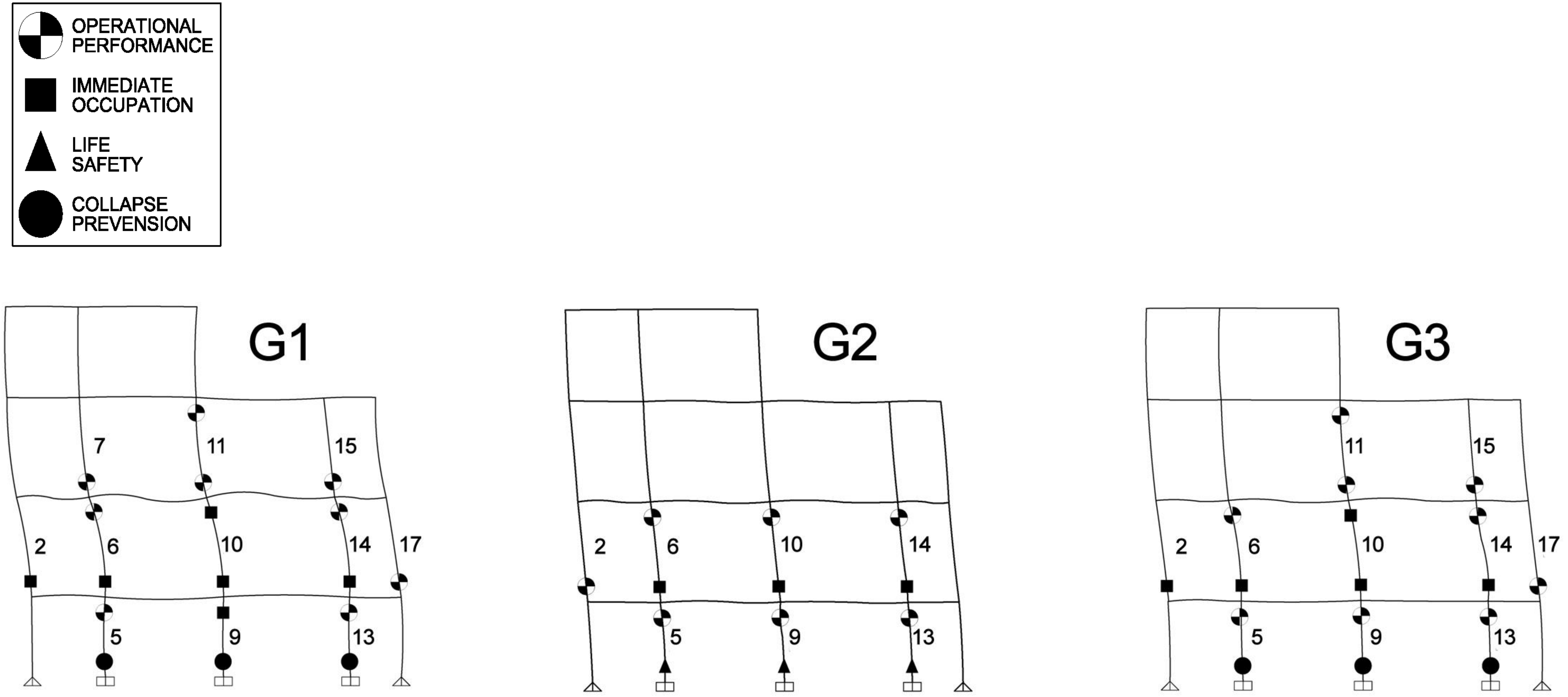


Table 6: Plastic formations and performance levels for columns of three groups

		GROUP 1 (G1)					GROUP 2 (G2)					GROUP 3 (G3)				
		Moments	Rotation	FEMA 356 limits (rad)			Moments	Rotation	FEMA 356 limits (rad)			Moments	Rotation	FEMA 356 limits (rad)		
Mem.	Joint	(kNm)	(rad)	IO	LS	CP	(kNm)	(rad)	IO	LS	CP	(kNm)	(rad)	IO	LS	CP
2	2	147.25	*0.0033*	**0.0021**	0.0166	0.0250	145.51	*0.0002*	0.0021	0.0166	0.0250	146.95	*0.0028*	**0.0021**	0.0166	0.0250
5	6	215.03	*0.0333*	0.0024	0.0188	**0.0282**	209.91	*0.0251*	0.0024	**0.0188**	0.0282	213.35	*0.0306*	0.0024	0.0188	**0.0282**
5	7	195.86	*0.0022*	0.0024	0.0188	0.0282	194.82	*0.0005*	0.0024	0.0188	0.0282	195.00	*0.0008*	0.0024	0.0188	0.0282
6	7	201.10	*0.0107*	**0.0024**	0.0188	0.0282	199.66	*0.0084*	**0.0024**	0.0188	0.0282	201.35	*0.0111*	**0.0024**	0.0188	0.0282
6	8	195.94	*0.0023*	0.0024	0.0188	0.0282	194.71	*0.0004*	0.0024	0.0188	0.0282	195.83	*0.0021*	0.0024	0.0188	0.0282
7	8	194.85	*0.0005*	0.0024	0.0188	0.0282	-	-	-	-	-	-	-	-	-	-
9	11	215.13	*0.0335*	0.0024	0.0188	**0.0282**	210.23	*0.0255*	0.0024	**0.0188**	0.0282	213.79	*0.0313*	0.0024	0.0188	**0.0282**
9	12	197.30	*0.0045*	**0.0024**	0.0188	0.0282	195.64	*0.0018*	0.0024	0.0188	0.0282	195.35	*0.0015*	0.0024	0.0188	0.0282
10	12	200.50	*0.0097*	**0.0024**	0.0188	0.0282	200.36	*0.0095*	**0.0024**	0.0188	0.0282	201.25	*0.0109*	**0.0024**	0.0188	0.0282
10	13	198.23	*0.0061*	**0.0024**	0.0188	0.0282	195.88	*0.0023*	0.0024	0.0188	0.0282	197.08	*0.0042*	**0.0024**	0.0188	0.0282
11	13	195.68	*0.0019*	0.0024	0.0188	0.0282	-	-	-	-	-	195.07	*0.0009*	0.0024	0.0188	0.0282
11	14	195.10	*0.0009*	0.0024	0.0188	0.0282	-	-	-	-	-	194.69	*0.0003*	0.0024	0.0188	0.0282
13	16	214.80	*0.0329*	0.0024	0.0188	**0.0282**	209.77	*0.0248*	0.0024	**0.0188**	0.0282	213.18	*0.0303*	0.0024	0.0188	**0.0282**
13	17	194.69	*0.0004*	0.0024	0.0188	0.0282	194.82	*0.0005*	0.0024	0.0188	0.0282	194.71	*0.0003*	0.0024	0.0188	0.0282
14	17	200.50	*0.0097*	**0.0024**	0.0188	0.0282	199.43	*0.0080*	**0.0024**	0.0188	0.0282	200.97	*0.0105*	**0.0024**	0.0188	0.0282
14	18	195.49	*0.0016*	0.0024	0.0188	0.0282	194.88	*0.0007*	0.0024	0.0188	0.0282	195.38	*0.0014*	0.0024	0.0188	0.0282
15	18	194.85	*0.0005*	0.0024	0.0188	0.0282	-	-	-	-	-	194.71	*0.0004*	0.0024	0.0188	0.0282
17	21	145.74	*0.0005*	0.0021	0.0166	0.0250	-	-	-	-	-	146.06	*0.0011*	0.0021	0.0166	0.0250



Plastic formations and performance levels for columns of three groups



Global level evaluation

The **transient and permanent** inter-story drift ratios are investigated (FEMA 356) Peak values are illustrated for all groups in Figure 4.

- All groups are in **LS level**.
- The maximum **transient inter-story drift ratios** (Fig 4a)

  for G1 and G3 occurred at *first story level*

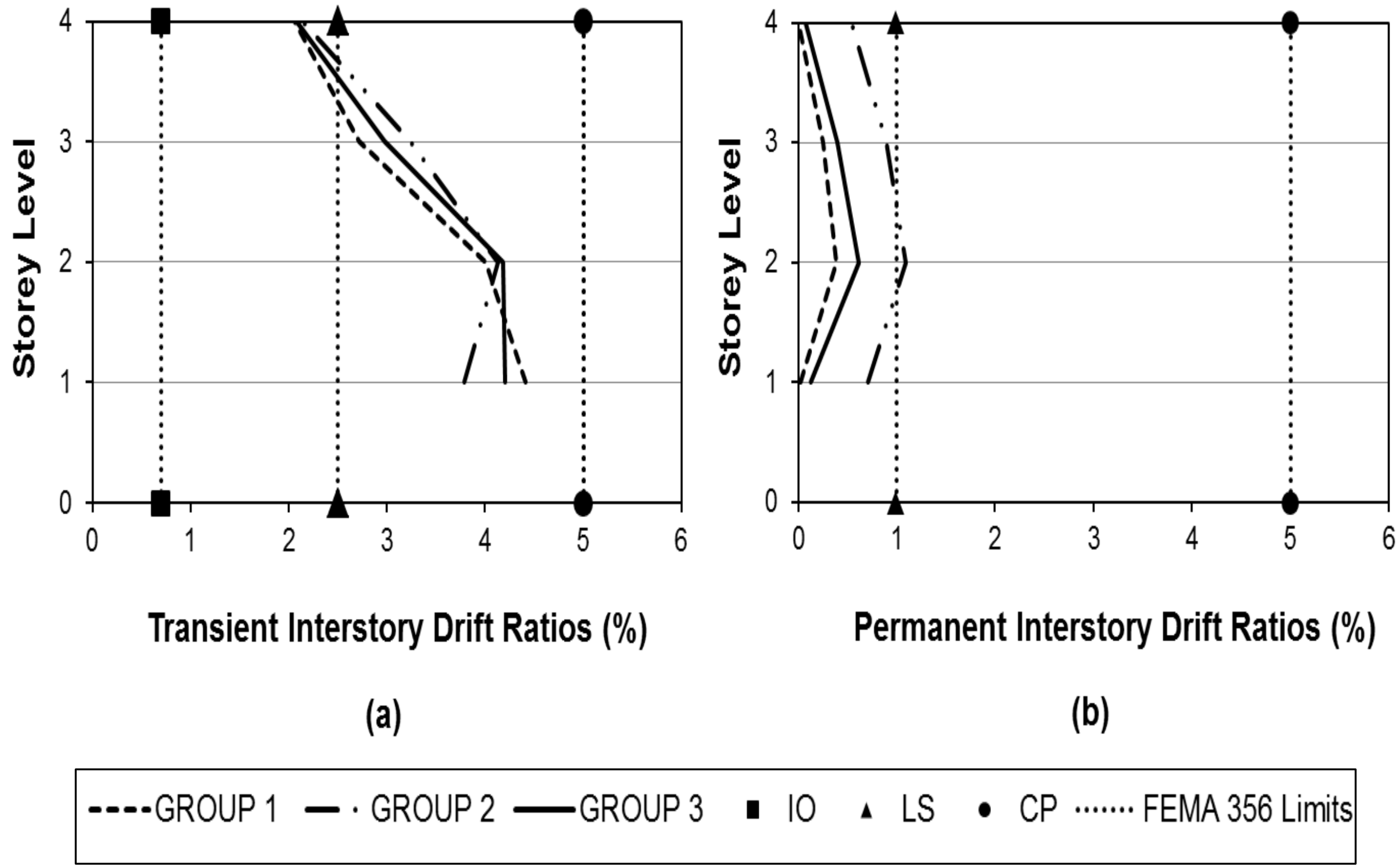
  for G2 occurred at *second story level.*

The ratio for G1, G2 and G3 are 4.4%, 4.13% and 4.20%, respectively.

The structure reached the **plastic limits** and **permanent drifts** were occurred (Fig 4b)

Permanent drift ratios did not exceed the LS limit for **G1 and G3** while **G2** is in the LS limit.

Maximum transient and permanent inter-story drift ratios of three groups



Conclusion

Conclusion

The **primary aim** of this research was

to investigate the performance of 2-D moment frames, in terms of strength, stiffness and ductility, using ***HSS and mild steel*** *end-plate beam to column connections.*

1. A typical 2-D steel moment frame from an existing residential building was taken as a case study.
2. **FEM** was used to capture the *moment curvature relationship* for the purpose of finding *hinge properties* in SAP2000 .
3. The **time-history** analysis (dynamic analysis) was carried out according to FEMA 356 to determine the **global inelastic response** of 2-D moment frames.

It is clear from the findings of this study that *more research is needed on HSS* to help in the *development of a guideline* to be added in the existing design codes.

Conclusion

The following are the main conclusions of this study.

1) Comparison of **hinging patterns** of three groups

   G1 models with 20 mm thick mild plate **11 hinges** yielded

   G2 models with 8 mm thick mild plate **12 hinges** yielded

   G3 models with 8 mm thick HSS plate **14 hinges** yielded

Conclusion

2) Reducing the mild steel end-plate thickness marginally **(G2)**

- improved the ductility of the joints
- enhanced the behaviour of the structural members

However,

- may cause *increase in lateral drift* of the joints
- may *lead to instability* of the frame.

On the other hand, reducing the end-plate thickness together with increasing the yield strength to HSS

- may provide a *better control of the stability* of the frame.

Conclusion

3) The **Moment-rotation behaviour** of the three groups indicate that

- the plastic hinge locations of the G1 models behaved as designed,

  the *strong column and weak beam* variation was achieved.

- G3 models with HSS did not achieve this in some cases where the damage or *failure occurs at the end-plate and bolts*.

- The ***thickness of the HSS end-plate*** can be the key for determining the **failure mode**.

4) The *usage of HSS* would allow the use of a **thinner end-plate** without compromising the frame seismic behaviour.

   However,

   the *minimum thickness* for the HSS end-plate may need to be determined to avoid buckling instability of the frame.

Thank you for listening