Design of a Cold Formed Steel Structure

May 30, 2018

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Who Are We?

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Özgenur Baştuğ

Utku Gürler

Can Özkan

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Agenda

01	Introduction to Cold Formed Steel Preliminary Design	• What is cold formed steel and where is it used? • What are the advantages? • Architectural Design • Lab Experiment
02	Cost Schedule Resources	• Cost Estimation • Schedule • Resources Used in the Project
03	Structural Frame Design Modelling of the Structural System	• Modelling the frames with SAP2000 • C and U sections & section properties • Structural modelling process
04	Earthquake, Dead & Live Load Calculations Capacity Calculations	• Load calculations • Dynamic properties (taken from SAP2000) • Demand / shear • Capacity calculations
05	Anchorage Design	• Outer Hold-down • Inner Hold-down

Cold Formed Steel Villa Project

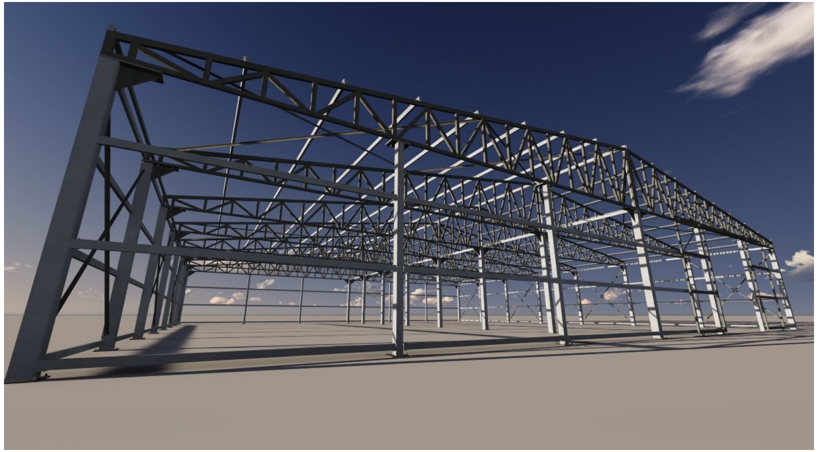
The project aims to design a two-storey villa at a seismic belt. According to the project info the land is provided and the team only needs to design the superstructure.

Project Constraints:

1. Cost
2. Time
3. Seismic belt

BOĞAZİÇİ ÜNİVERSİTESİ 1863

What is Cold Formed Steel?



- Cold formed steel (CFS) is a type of steel which is made by rolling or pressing at relatively cold temperatures.

- CFS members are produced using structural quality sheet steel.

- No heat is required for its formation and various thicknesses of steel frames are available for various uses.

- Cold-formed steel is generally used in the constructions of residential buildings not exceeding 3 or 4 floors.

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History of CFS

- The use of CFS in construction began in the 1850s.
- First documented use of CFS: around 1925.
- In 1920s and 1930s, limited acceptance due to lack of adequate design standard.

CFS Around the World

- CFS is highly used in the USA, Scandinavian Countries, Western Europe, Japan and Australia.
- In the USA, CFS usage is 25% and in Japan this rate is 15%
- The usage of CFS in Turkey is believed to be limited to 0.5%

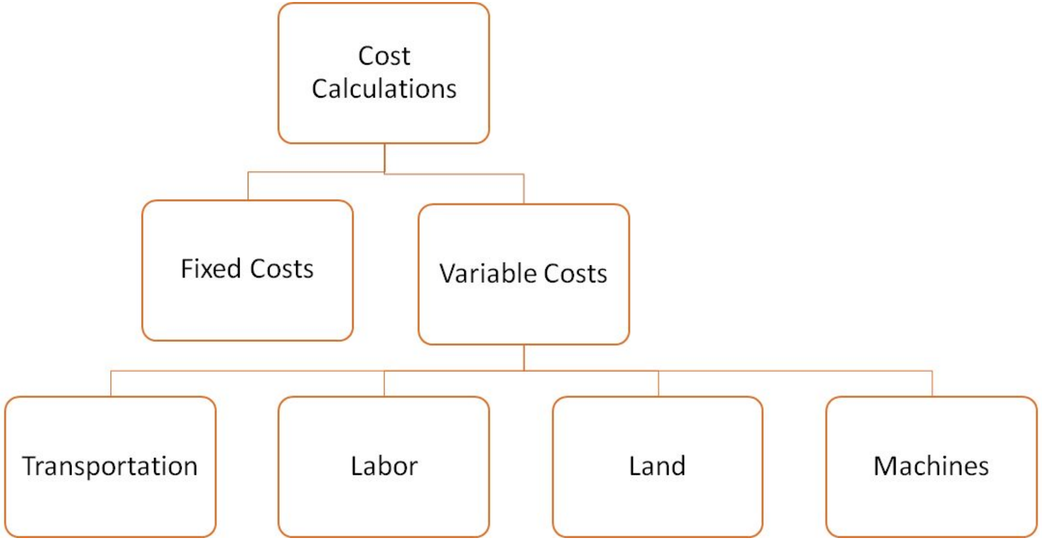
Advantages

- Sustainability
- Durability
- Compactness
- Lightness
- High strength and stiffness
- Ease of fabrication and application
- Elimination of delays
- Economical transportation and handling

Disadvantages

- Thinner steel members
- Prone to local buckling
- High unit price
- Low fire resistance

Cost Analysis



- Cost of the projects for similar existing projects are utilized.
- Accurate cost estimation is hard.
- It has comparatively low cost variance.

Estimated Cost: 500,000TL

Schedule

Preliminary Design

Literature Review
Experiment Investigation
Architectural Design
Structural Member Selection
Cost/ Schedule/ Resource

02.2018

03.2018

Engineering Process

Frame Modelling with SAP2000
Structural Modelling with SAP2000
Load Calculations

Performance Assessment

Demand Calculations
Capacity Calculations
Capacity and Demand Comparison

04.2018

05.2018

Final Review

Second Iteration
Anchorage Design
Inspection of the Project Report

TODAY!

Final Presentation

30.05.2018

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Resources

SAP2000

SketchUp

AutoCAD

Google docs

smartsheet

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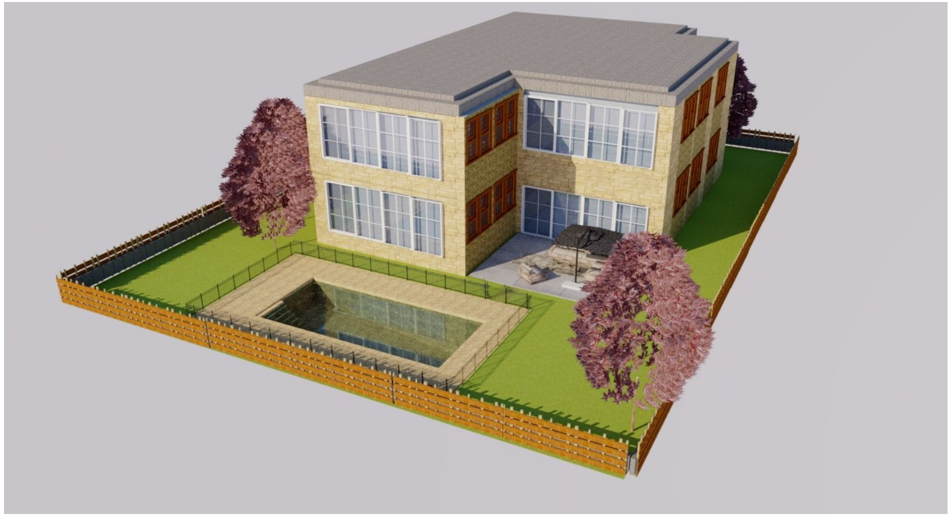
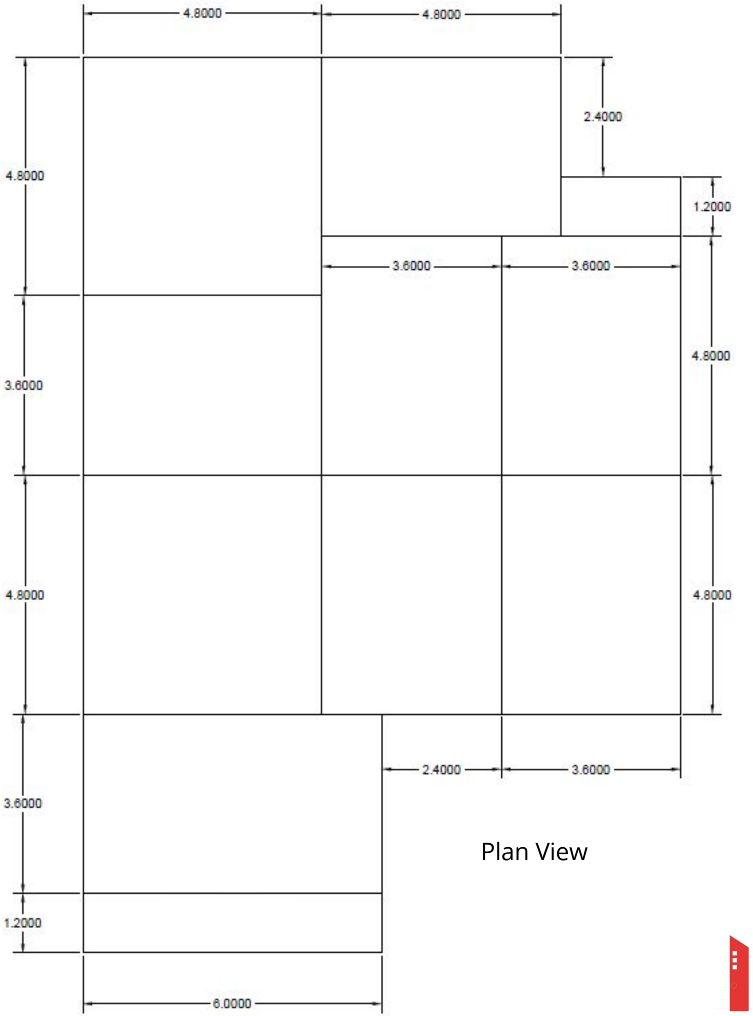


Architectural Design

- Cold-formed steel panel dimensions: 1.2m x 2.4m (most common dimensions)

- Design constraints:
  - Avoidance of excessive span lengths
  - Comfortable and practical residential housing system



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Common properties of CFS shear panels.

Common properties	Panel height (mm)	Panel width (mm)	Sheathing thickness (mm)	Screw type (mm)
	2400	1200	12.5	4.2 * 16 flat head screws

Sheathing board and cold-formed steel properties.

Properties	Modulus of elasticity, *E* (MPa)	Weight per unit volume (kg/m^3)	Poisson's ratio
Board type 1	3053 in short direction 2404 in long direction	640	0.167
Board type 2	4009	880	-
CFS	203,395	7849	0.3

St37 grade steel | | nominal yield strength fy = 227.5 MPa
nominal tensile strength ft = 310.3 MPa

Lab Experiments

Lab experiments on cold formed steel made by Assoc.Dr.Serdar Soyöz and Burak Karabulut.

Origin of our current project
Same structural material properties
In contrast with the experiment, linear analysis is made.

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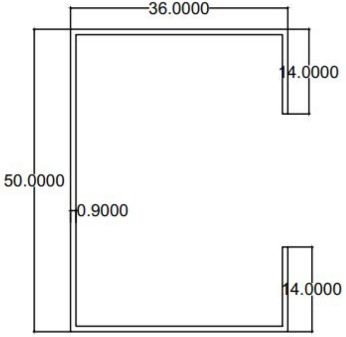
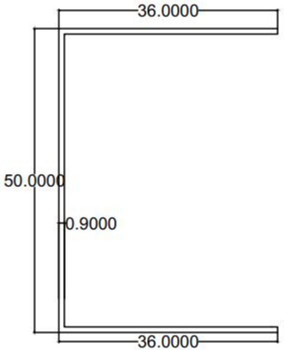
Structural Frame Modelling

Structural frame modelled as:

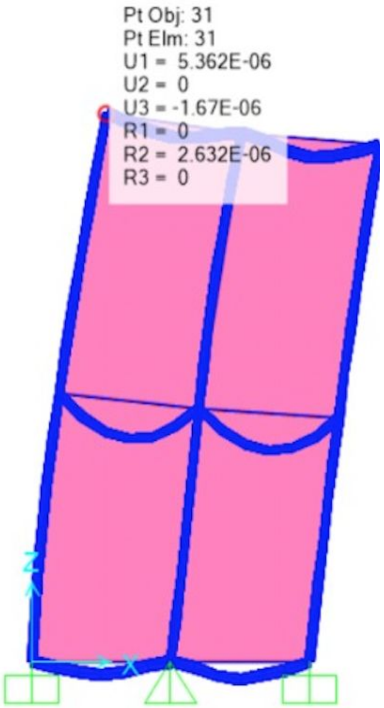
- 1.2 x 2.4 m panel dimensions, taken from the standards
- C section outer studs, C section mid-stud, U section lateral members
- 4 boards attached to the frame from 9 points
- Shear loads are not allowed on frame, shear is carried by the sheathing material only.



C Section



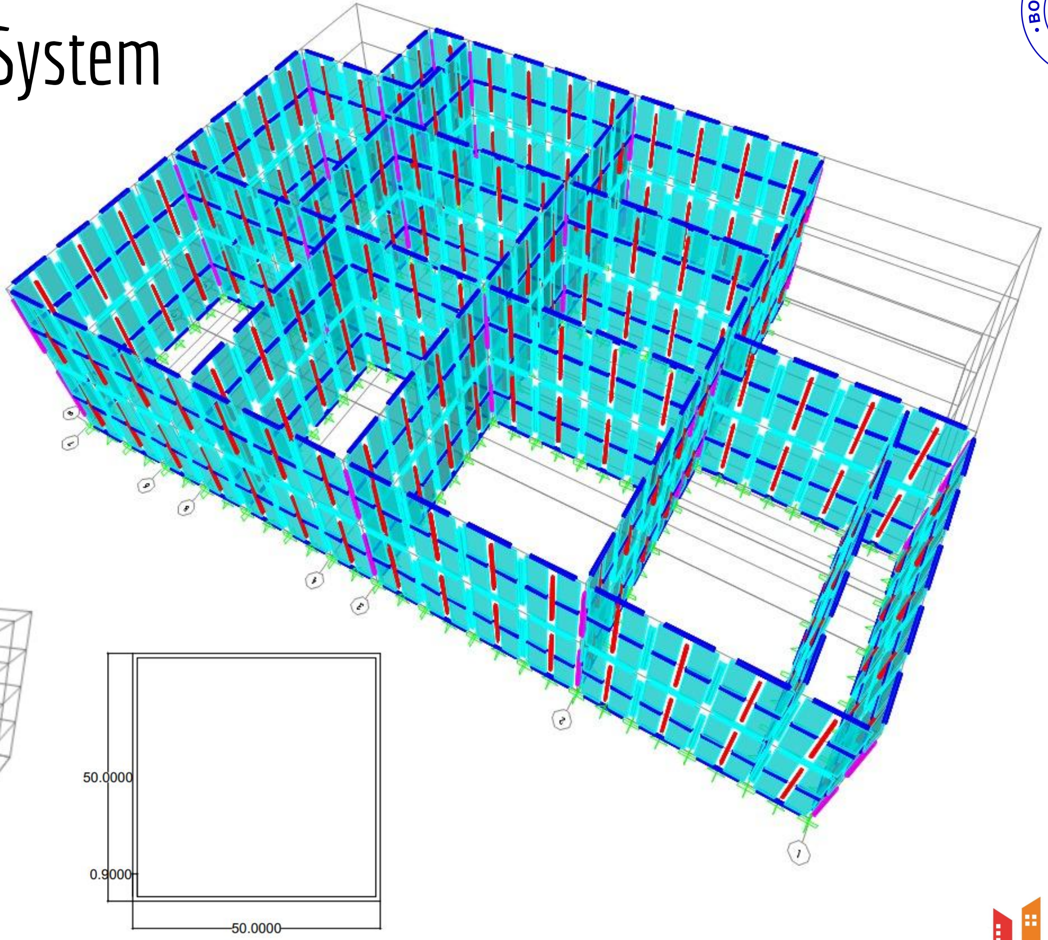
U Section



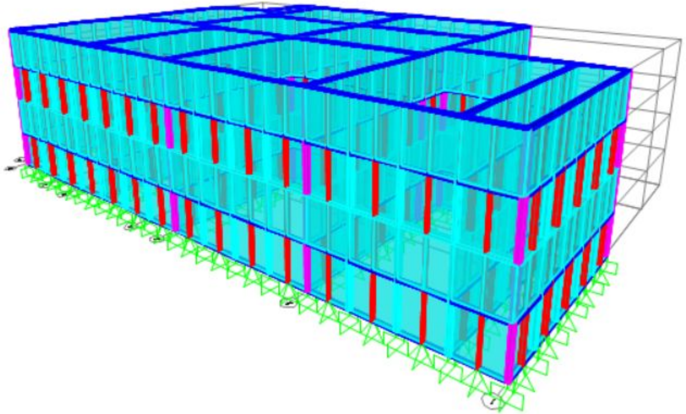
Lateral displacement under force F = 0.02 kN, thickness t = 25mm

Modelling of the Structural System

- Replication of frames
- Box sections: Solution for wall corners!
- Slab members are inserted as masses on diaphragm



Box Section



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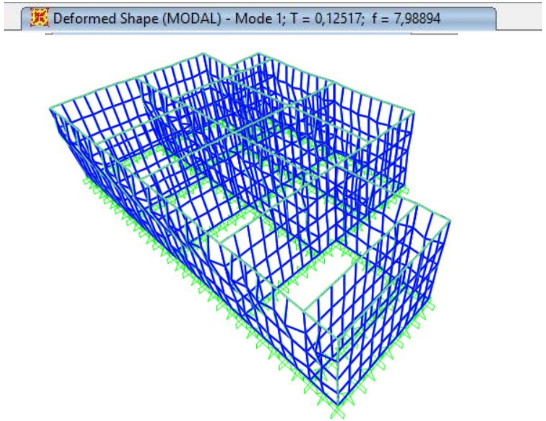
Modelling of the Structural System

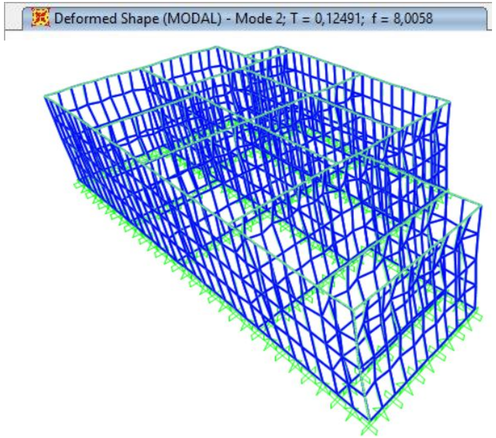
Modal Analysis

- Modal analysis uses the overall mass and stiffness of the structure to find the various periods at which the structure will naturally resonate.
- It is also used to for detecting connection errors in a SAP2000 model.

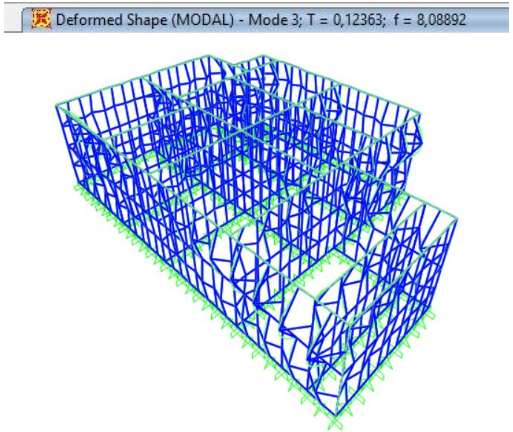
Period
T = 0.12



First Mode

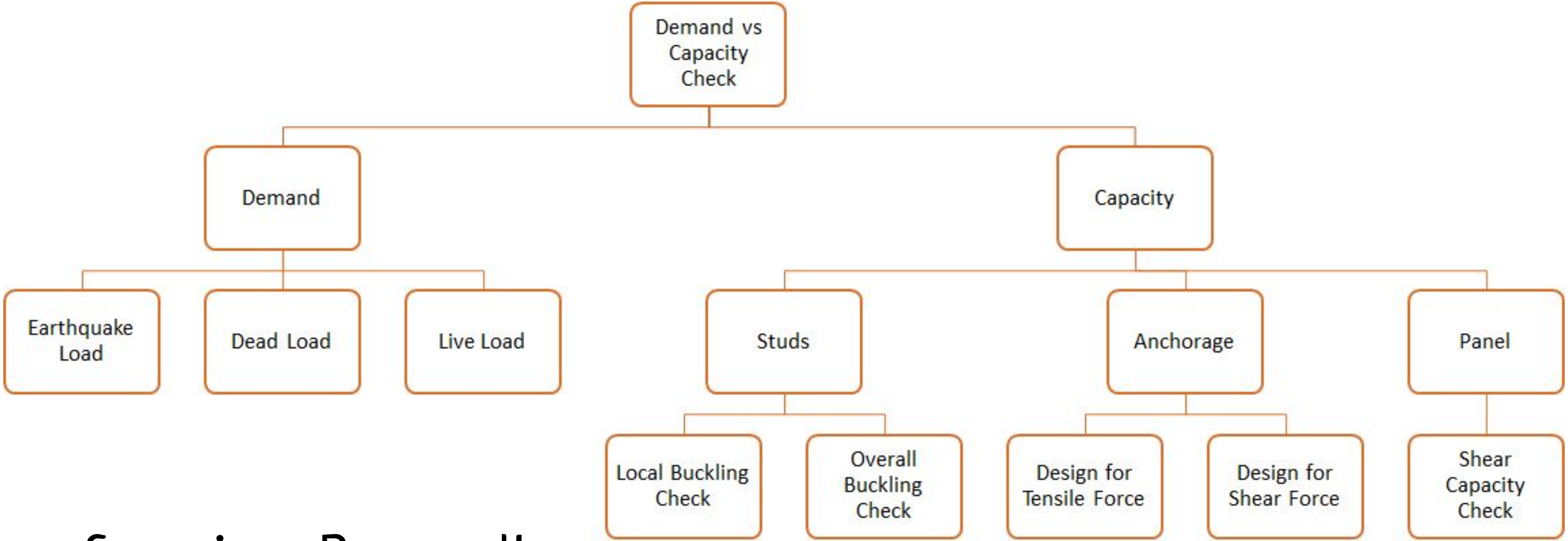


Second Mode



Third Mode

Performance Assessment



Capacity › Demand!

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Load Calculations

Dead Load Calculations

- Board Weight (with rock wool)
- C-Section
- U-Section
- Slab Weight

Live Load Calculations

Load calculations are made according to TS498 and Earthquake Code (2007).

- The live load for residential buildings is taken as 2 kN/m^2 from the code.

Total Dead Load: **620.89 kN**

Total Live Load: **725.26 kN**

Usage				Calculation Value
	ROOFS Lateral or inclined up to 1/20	Slabs	STAIRCASES (Including landing and staircase entrance)	kN/m^2
1		Loft rooms		1,5
2	Occasionaly used roofs	Housing, terrace room and corridors, offices, shops at the residences up to 50 m2, hospital rooms		2

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Earthquake Load Calculations:

- Base Shear Force

$$V_t = \frac{W A(T_1)}{R_a(T_1)} \geq 0.10\, A_o\, I\, W$$

- W: total weight of the building
- A(T): spectral acceleration coefficient
- R_a: earthquake load reduction coefficient

- Spectral Acceleration Coefficient

$$A(T) = A_o\, I\, S(T)$$

- A_0: effective ground acceleration coefficient
- I: building importance coefficient
- S(T): spectrum coefficient

Earthquake Load Calculations:

Seismic Zone	A_o
1	0.40
2	0.30
3	0.20
4	0.10

Purpose of Occupancy or Type of Building	*Importance Factor (I)*
4. Other buildings Buildings other than above defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)	1.0

Local Site Class according to Table 6.2	T_A *(second)*	T_B *(second)*
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

$$S(T) = 1 + 1.5\frac{T}{T_A} \qquad (0 \le T \le T_A)$$

$$S(T) = 2.5 \qquad (T_A < T \le T_B)$$

$$S(T) = 2.5\left(\frac{T_B}{T}\right)^{0.8} \qquad (T_B < T)$$

Earthquake Load Calculations:

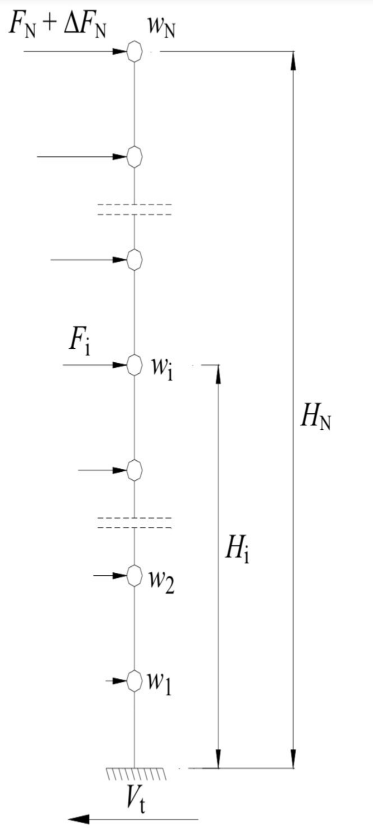
- Total Earthquake Load: **320.61 kN**

- Distribution of earthquake load to each storey

$$V_t = \Delta F_N + \sum_{i=1}^{N} F_i$$

$$F_i = (V_t - \Delta F_N) \frac{w_i H_i}{\sum_{j=1}^{N} w_j H_j}$$

$$\Delta F_N = 0.0075 \, N \, V_t$$



Demand

Frame Text	Station m	OutputCase	CaseType Text	P KN	V2 KN	V3 KN
721	0	negative x d...	LinStatic	-11,054	0,0003697	0,
721	1,2	negative x d...	LinStatic	-11,024	0,0003697	0,
775	0	x direction	LinStatic	-10,666	-0,0002732	0,
775	1,2	x direction	LinStatic	-10,637	-0,0002732	0,
1126	0	x direction	LinStatic	-9,986	-0,0007981	0,0
1126	1,2	x direction	LinStatic	-9,956	-0,0007981	0,0
1025	0	negative y d...	LinStatic	-9,14	-5,73E-12	-0,
1025	1,2	negative y d...	LinStatic	-9,11	-5,73E-12	-0,
18	[illegible]	negative x d...	LinStatic	[illegible]044	1,505E-12	0,
18	[illegible]	[illegible]	[illegible]	[illegible]014	1,505E-12	0,
1076	[illegible]	[illegible]	[illegible]	[illegible]984	-3,712E-12	-0,
1076	1,2	negative y d...	LinStatic	-8,954	-3,712E-12	-0,
1019	0	negative x d...	LinStatic	-8,894	9,15E-20	0,
1019	1,2	negative x d...	LinStatic	-8,864	9,15E-20	0,

Calculated loads are assigned to the model.

- Slabs have mass, therefore even if they are not modeled, their weight must be assigned to the model!
- Dead and live loads on slabs were distributed onto beams using tributary area method.
- 1.2D+1.0Q+1.0Ex+0.3Ey & 1.2D+1.0Q+1.0Ey+0.3Ex load cases are used.

According to these loadings,

Maximum demand for axial load:

5.53 kN

Maximum demand for shear load:

6.41 kN

Capacity Calculations

Local Column Buckling

- Flexural Buckling

$$KL = 2 \times \text{spacing of screws} \qquad F_e = \frac{\pi^2 E}{(KL/r)^2}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad F_n = (0.658^{\lambda_c^2}) F_y$$

- Torsional–Flexural Buckling

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right] \qquad \sigma_t = \frac{1}{Ar_0^2}\left[GJ + \frac{\pi^2 E C_w}{(KL)^2}\right]$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad \beta = 1 - \left(\frac{x_0}{r_0}\right)^2$$

$$F_n = (0.658^{\lambda_c^2}) F_y \qquad \sigma_{ex} = \frac{\pi^2 E}{(KL/r_x)^2}$$

Capacity Calculations

Overall Column Buckling

- ➔ Flexural Buckling
- ➔ Torsional-Flexural Buckling

$$\sigma_{CR} = \sigma_{ey} + \bar{Q}_a \qquad \bar{Q} = \bar{Q}_o\left(2 - \frac{s}{s'}\right)$$

$$\sigma_{ey} = \frac{\pi^2 E}{(KL/r_y)^2}$$

$$\sigma_{ex} = \frac{\pi^2 E}{(KL/r_x)^2} \qquad \sigma_t = \frac{1}{Ar_0^2}\left(GJ + \frac{\pi^2 EC_W}{L^2}\right) \qquad \bar{Q}_t = \frac{\bar{Q}d^2}{4Ar_0^2}$$

$$\sigma_{tQ} = \sigma_t + \bar{Q}_t \qquad \sigma_{CR} = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_{tQ}) - \sqrt{(\sigma_{ex} + \sigma_{tQ})^2 - 4\beta\sigma_{ex}\sigma_{tQ}}\right]$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad F_n = (0.658^{\lambda_c^2})F_y$$

Sheathing Parameters from AISI Specification

Sheathing(2)	k	$\bar{Q}_o$ kN	$\bar{\gamma}$ length/length
3/8 in. (9.5 mm) to 5/8 in. (15.9 mm) thick gypsum	24.0	107.0	0.008
Lignocellulosic board	12.0	53.4	0.009
Fiberboard (regular or impregnated)	7.2	32.0	0.007
Fiberboard (heavy impregnated)	14.4	64.1	0.010

Capacity Calculations

Local Column Buckling

Flexural Buckling					Torsional-Flexural Buckling		
Around X-Axis	K * L (mm)	600			β	0,305	
	K*L / rx	29,28			σ ex (Mpa)	2341,826	
	Fe (Mpa)	2341,826			σ t (Mpa)	486,312	
	Fy (Mpa)	227,5			Fe (Mpa)	421,862	
	λc	0,312			Fy (Mpa)	227,5	
	Fn (Mpa)	218,435	<1.5	Fn = (0.658^(λc^2)) * Fy	λc	0,734	<1.5
					Fn (Mpa)	181,533	
Around Y-Axis	K * L (mm)	600					
	K*L/ry	42,03					
	Fe (Mpa)	1136,135					
	Fy (Mpa)	227,5					
	λc	0,447	<1.5				
	Fn (Mpa)	209,210					

Overall Column Buckling

σCR Calculation (Flexural)			
Around X-Axis	σ ex (Mpa)	146,364	
	Qo (N)	107000	
	s (mm)	300	
	s' (mm)	300	
	Q (N)	107000	
	Qa (Mpa)	816,1709	
	σCR (Mpa)	962,535	
Around Y-Axis	σ ey (Mpa)	71,008	
	Qo (N)	107000	
	s (mm)	300	
	s' (mm)	300	
	Q (N)	107000	
	Qa (Mpa)	816,1709	
	σCR (Mpa)	887,179	
	λc	1,473	<1.5
	Fn (Mpa)	91,773	GOVERNS

σCR Calculation (Torsional - Flexural)			
Around X-Axis	β	0,305	
	σ ex (Mpa)	146,364	
	σ t (Mpa)	40,117	
	Qt	249,195	
	σ tQ (Mpa)	289,312	
	σCR (Mpa)	104,887	governs

USE SMALLER σCR as Fe (Fe = σ CR)

Nominal & Allowed Load Calculations

	Nominal Axial Load		Allowable Axial Load	
(Cross sect. Area)	Ag (mm2)	131,1		
(Effective Area Coeff.)	k	0,8		
(Effective Area)	Ae (mm2)	104,88		
			ϕc	0,85
(Nominal Stress)	Fn (Mpa)	91,773		
(Nominal Axial Load)	Pn (N)	9625,153	Pn (N)	8181,380
(Nominal Axial Load)	Pn (kN)	9,625	Pn (kN)	8,181

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Capacity Calculations

Shear Capacity of Frame

$$V_c = \phi v_c \sum l_i$$

Design Method		Earthquake
ASD	Ω	2,5
LRFD	Φ	0,6

- Characteristic strength of sheathing is taken as **5 kN/m**.
- **2** sheathings with **1.2m** width at each side are used in each frame.
- Shear capacity of each frame is found to be **7.2 kN**.
- LRFD method is used to calculate the design strength.

$$P_u = \phi_c P_n$$

- Effective area is calculated by a multiplier method previously used by MIT.

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Anchorage - Outer Hold-down

(M12 > 80 mm)

Tensile Capacity

- ➔ Steel Failure Capacity → Directly calculated due to ETAG 0005 → **28.06 kN > 25.87 kN**
- ➔ Pull-out Failure Capacity → Directly calculated due to ETAG 0005 → **Not decisive**
- ➔ Concrete Cone Failure → Calculated due to formulas given in ETAG 001 & values given in ETAG 0005 → **26.37 kN > 25.87 kN**

Shear Capacity

- ➔ Steel Failure Capacity → Directly calculated due to ETAG 0005 → **21.92 kN > 8.8 kN**
- ➔ Pull-out Failure Capacity → Directly calculated due to ETAG 0005 → **52.74 kN > 8.8 kN**

Anchorage - Inner Hold-down

(M10 > 65 mm)

Tensile Capacity

➔ Steel Failure Capacity	Directly calculated due to ETAG 0005	**17.33 kN > 16.33 kN**
➔ Pull-out Failure Capacity	Directly calculated due to ETAG 0005	**Not decisive**
➔ Concrete Cone Failure	Calculated due to formulas given in ETAG 001 & values given in ETAG 0005	**17.71 kN > 16.33 kN**

Shear Capacity

➔ Steel Failure Capacity	Directly calculated due to ETAG 0005	**10.96 kN > 8.8 kN**
➔ Pull-out Failure Capacity	Directly calculated due to ETAG 0005	**17.71 kN > 8.8 kN**

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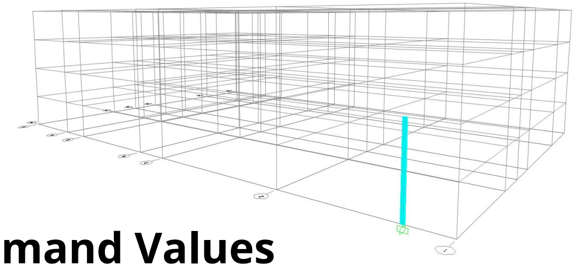
Capacity vs. Demand

The Most Critical Stud



Capacity Values

Stud: 8.18 kN

Panel: 7.2 kN

Anchorage: 26.37 kN (M12)

& 17.33 kN (M10)

Demand Values

Stud: 5.33 kN

Panel: 6.41 kN

Anchorage: 25.87 kN (M12)

& 16.33 kN (M10)

Capacity › Demand!

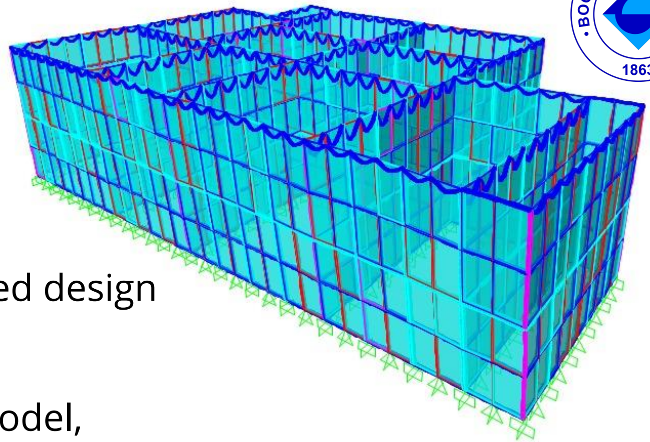
Iteration

- It must be noted that these results were the consequence of a continuous iteration process.

- The aim of the iteration was:
    - To reach a more economical design

	First Iteration	**Last Iteration**
Capacity (kN)	20,82	8,18
Demand (kN)	7,3	5,33
Web (mm)	99	50

Conclusion



The Deformed Shape Under Most Critical Loading Case

- CFS is a relatively new application in Turkey and computer aided design of CFS is rather ambiguous.
- It was challenging to model and analyze a cold formed steel model, especially with limited software and application.
- Turkish Code did not even have CFS steel category until the latest release, it was treated as regular steel and modified with multipliers!
- Contemporary calculations are usually done by hand.
- New automated analysis methods are needed to streamline the CFS design process!

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