ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE ENGINEERING AND TECHNOLOGY

PONDING STABILITY ON SPACE TRUSS ROOF SYSTEM

M.Sc. THESIS

Aslı ARAS

Department of Civil Engineering

Structure Engineering Programme

OCTOBER, 2016

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ISTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

UZAY KAFES ÇATI SİSTEMDE GÖLLENME STABİLİTESİ

YÜKSEK LİSANS TEZİ

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501121005

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Tez Danışmanı: Doç. Dr. Elişan Filiz PİROĞLU

EKİM, 2016

Aslı Aras, a M.Sc. student of ITU Graduate School of Engineering, Science and Technology student ID 501121005, successfully defended the thesis entitled "**PONDING STABILITY ON SPACE TRUSS ROOF SYSTEM**", which she prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

Thesis Advisor :	**Doç. Dr. Elişan Filiz PİROĞLU** ISTANBUL Technical University
Jury Members:	**Yrd.Doç. Dr. Kadir ÖZAKGÜL** ISTANBUL Technical University
	Yrd.Doç. Dr. Devrim ÖZHENDEKÇİ Yıldız Technical University

Date of Submission : 15 September 2016
Date of Defense : 19 October 2016

To my family,

FOREWORD

I owe special thanks to my advisor Doç.Dr. Filiz Piroğlu since she really acted as passioned and tolerated me so many times throught this study.

October 2016

Aslı ARAS
(Civil Engineer)

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ABBREVIATIONS

ASCE : American Society of Civil Engineers
AISC : American Institute of Steel Construction
SEI : Structural Engineerins Institue
IBC : International Building Code
SJI : Steel Joist Institue
IPC : International Plumbing Code
BS : British Standards
NIST : National Institute of Standards and Technology
CAST : Collapse Analysis of Space Trusses

SYMBOLS

$\sigma_{crit.}$	**:** Critical strees
ε	**:** Elasticity modulus
M_x, M_y, M_{xy}	**:** Moment components
N_x, N_y, N_{xy}	**:** Normal force components
q	**:** Phase load
t	**:** Time
u,v	**:** Displacement vector components

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PONDING STABILITY ON SPACE TRUSS ROOF STRUCTURE

SUMMARY

Ponding stability which subject is inserted into the code of steel structure design and construction drafted in our country in these days, is main concern of this study.

In Chapter 3.4 at Ponding Stability topic, literature review is done about the accumulation of water in the roof system to have sufficient strength and rigidity against the action to the requirements. In 1962, Haussler is the first man who come up with the ponding stability idea. After his calculations, resarchers continue their studies up to present and some changes observed like changes in assumptions and changes in empirical formulas. In Chapter 3, causes and prevention methods of ponding is handled, too.

The requirements of the roof drainage system to prevent ponding effect is identified in codes. According to the codes, there should be two system to work controlled. One is siphonic system located on the roof, the other is scuppers located at the level of roof membrane on the parapet wall. It means, in case of siphonic system failed, scuppers work properly. Height of the parapet wall that is concern for the architects can not be ignored since it has importance for the working principle of scuppers.

In Chapter 5, to see how ponding triggers the roof members to loose their stiffness and strength, numerical analysis is done on the steel roof which is nearly flat. The alternate method is used for both linear and nonlinear static anaylsis.

UZAY KAFES ÇATI SİSTEMDE GÖLLENME STABİLİTESİ

ÖZET

Ülkemizde çelik yapıların tasarım ve yapım kuralları standartının taslak olarak hazırlandığı bu günlerde, bu taslak çalışmaya dahil olan çelik çatı dizaynın da gözardı edilemeyecek olan göllenme etkisi bu çalışmanın ana konusudur.

Çatı sisteminin su birikmesi etkisine karşı yeterli dayanım ve rijitliğe sahip olması için gereklilikleri kapsamında literatür taraması yapılarak Haussler' in ilk çalışmasından yani 1962' ler den bugüne hesaplamalarda görülen amprik değişimler yapılan kabuller Bölüm 3.4' te Göllenme Stabilitesi başlığında ele alınmıştır.Yine bölüm 3' te göllenmenin nedenleri ve göllenmeyi önleme yöntemleri üzerinde durulmuştur.

Göllenme etkisinin önlenmesi için çatı drenaj sisteminin gereklilikleri de standartlarda tariflenmiştir.Buna göre sifonik sistemlerin tam verimle çalışamadığı tıkanma, yenilenme zamanın gelmesi gibi durumlarda eski yapılarda daha çok kullanılan çörtenlerin devreye girmesi gerekliliği ve mimari detaylar oluşturulurken parapet yüksekliğinin drenaja etkisinin de gözardı edilmemesinin önemli olduğu saptanmıştır.

Bölüm 5' te nümerik analiz kısmında eğimi düşük bir çelik çatıda çalışmayan bir drenaj sisteminde göllenmenin çatı elemanlarının dayanım ve rijitliklerini nasıl kaybettirdiği linear ve non- linear hesaplar yapılarak gösterilecektir.

1. INTRODUCTION

Structural engineers design, create, analyze and solve problems, as well as innovate and use maths combining with science to shape the world. People use the structures created by structural engineers in every part of their daily life, from houses, to theatres, sports stadia and hospitals, airports, bridges and industrial buildings. Structural engineers are involved at every stage of a structure's realization, and they play a key part in design. Working with architects, and alongside other professional, structural engineers create conceptual designs by examining structures at risk of collapse and advising how to improve their structural integrity and guarantee that the structure is safe to build.

Structural engineers should perform structural analysis to understand the effects of loads/stresses caused by gravity, the users of the structure, and the widely varying climatic conditions and ground conditions around the world. In order to do best suiting structural analysis, the magnitudes of the various loads that are likely to be applied to the structure over its lifetime should be accurately. The probability of the simultaneous application of the various load types should be considered, too.

During the structural analysis, choosing appropriate load and their combinaton for the structure is also an important feature of the structural engineer's work. Professional judgement in structural engineering occurs in the course of time. Along the time, standards are always guidance for the engineers to learn the principles of the profession and to become professional.

The most commonly used standard is ASCE/SEI 7 Minimum Design Loads for Buildings and Other Structures in structural engineering includes wind, rain, snow, earthquake, live and dead loads. Also included in the ASCE Minimum Design Loads publication are loads due to hydrostatic loads, flood loads and earth pressure loads [1].

Building code provisions for design snow loads incorporate light rain on snow, but heavy rainfall is not included. Therefore, the structural engineer must consider this additional load separately. Factors influencing the rain-on-snow load include rain intensity, roof geometry, and drainage characteristics of the roof. Duration is also considered because continuous rain can wash away snow, effectively reducing the risk of snow-induced collapse. Conversely, a period of short rain may cause snow to melt and become further saturated, significantly increasing the load on the roof structure [1].

The design loads corresponding to the highest accumulation of snow load can be found from past weather records maintained by Meteorological Departments. If storm water is drained properly, rain does not contribute to any load on the structure. However, structural failures have occurred when rainwater has accumulated on roofs due to choked storm water drains. The accumulation of water causes additional load and hence deflection permits more water to accumulate. The progressive deflection and accumulation of water may continue, leading to structural failure [2]. This type of progressive deflection related roof collapses are common, destructive, and potentially life threatening. They often occur without warning, and can be difficult to predict [3].

When the continued accumulation of rainwater are large enough to overload the structure then it causes increasingly larger deflections, a condition known as "ponding instability" occurs.

This type of rain loading causing potential collapse of roof systems under ponding condition demand more research, a better understanding of the phenomena, and more prescriptive design requirements in building codes. To drawn attention about the ponding instability phenomena, what happens to space truss roof when ponding occurs. This subject is handled in this paper.

In this paper, there will be five main parts including; Space truss roof systems, definition, reason and precautions for ponding, overview of standards about the ponding and numerical analysis of the problem.

2. SPACE TRUSS ROOF SYSTEM

Double layer grid steel space truss system is commonly used form of the load carrying system for the roof stuructures, more than 25 years in Turkey. A space truss is a lightweight rigid structure consisting interlocking struts in a geometric pattern by using bolts, sleeves and connection nodes. It derives its strength from the inherent rigidity of the triangular frame. Flexural loads are only transmitted as tension and compression loads along the length of each strut. The system carries loads by axial forces. Since the space frame systems are three-dimensional structures, which work in two direction, for large spans, it provides economical solutions. Moreover, the design, manufacture, installation process is completed in a very short interval due to the use of prefabricated assemblies components. It gives a big opportunity to the customer to start his production. Transporting to far distances is easy due to the use of prefabricated components. Space frame systems are the most useful structures for the earthquake areas due to their light unit weight. Important components of the space truss joint are as follows: Pipe, weld, endcone, sleeve, bolt and meronode. As shown in Figure 2.1.

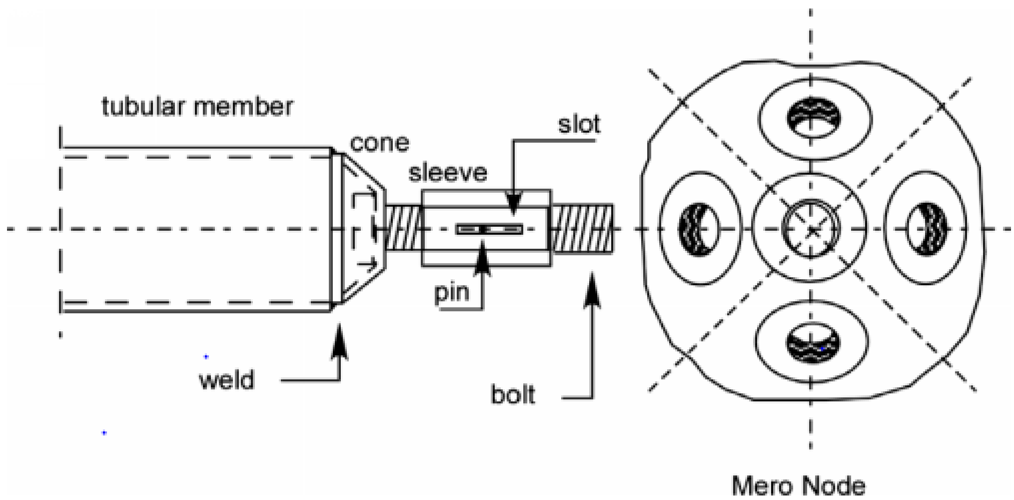


Figure 2.1 : Typical Connection Detail of Mero System [4].

Mero system consisting of a steel sphere with screw thread holes is used as joint assembly at the truss connection joints. Additionally chord and diagonal members having a steel tube sections with different diameters and thicknesses belong to the load bearing members of this Mero system, as shown in Figure 2.2. The screwed cone ends of these pipes are connected by the steel sphere joints. These spheres with flat faces and tapped holes for bolts are hot-pressed forging nodes, which are constituted as connectors, i.e. assemblies by tightening the bolts by means of hexagonal sleeve and dowel pin arrangement for the straight bearing circular hollow sections of the space truss system without causing any joint-eccentricity. At every node-point the axes of all the joining truss members can pass through the center of these connectors so that only axial forces can develop. Thus, tensile forces will be resisted by the space truss members and transmitted along the longitudinal axis of the bolts on their end cone. However, there will be no stress on the bolts if they are subjected to compression forces, which will be then distributed through the hexagonal sleeves to these connectors.

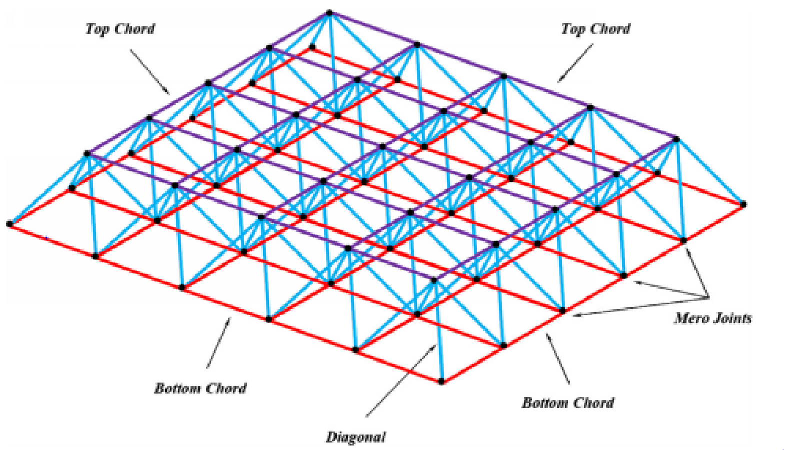


Figure 2.2 : Schematic View of a Mero Type Steel Space Roof Structure [5].

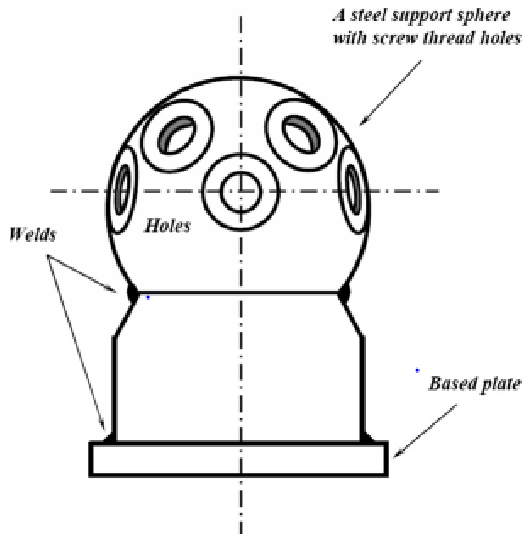


Figure 2.3 : Pattern of a Steel Support Sphere Welded on a Baseplate [5].

Even though space truss roof systems become very popular in last decades, stability problem may occurs due to rainwater ponding, easily. Deflection increases due to loss of stability under ponding load cause sudden failures on the roof structure. There are many examples to ponding failure on these roof structures, as seen in following Figure 2.4 and Figure 2.5.

Figure 2.4 : Collapse of a Department Store in Bolivar [6].

Figure 2.5 : Collapse of a Factory in Istanbul [7].

3. PONDING

3.1 Definition of Ponding

As underlined at the beginning of this study, the ponding condition can be defined simply as progressive deflection and resulting accumulation of load until either stability or collapse is reached. In a typical scenario, water will cause deflection. Assuming water is available, it can raise this deflection to a certain height (to at least the height of the supports), and the deflection will create a still larger volume for the water to fill in the roof. As more water flows in, the deflection increases, and the water level continues to rise. This process can continue to one of three end cases. First, the roof system could reach instability, in which case excess water will flow out over the edges, leaving collected water to eventually drain or dry out later. Second, the roof system could approach instability, but reach an overload condition before instability, and fail because the loads are too large. Third, in the most dangerous case, the roof deflections could rapidly become large and unbounded rapidly so that the roof system will never reach stability. In this case, the roof will fail eventually due to overload.

3.2 Causes of Ponding

Ponding loads can be caused either by rain or snow loads. It is common for snow on a roof to melt as heat passes through the building membrane, which can lead to the ponding effect. Additionally, snow on a roof often acts as a sponge, absorbing rainfall, and increasing the loads on a roof. Rain after a snowstorm may produce some of the heaviest loads that roof will experience, and it can lead to ponding.

Several things must be present in a roofing system for it to be susceptible to ponding loads. First, it must be a relatively flexible roof. Without this quality, the roof will not deflect enough to collect additional water to create a ponding situation. In addition, a roof either must be relatively flat, or sloped with some form of a parapet that allows collection of runoff water. Each of these properties will allow water to pond, and initiate deflections on structures that may continue to failure. Other issues that can exacerbate the problem include blocked, misplaced, or missing drains or scuppers, and initial sag due to mechanical units or other unexpected dead loads. One problem to be aware of is that often, drains are placed near columns [8]. This can be a problem because as the roof deflects under load, the points at columns will be the high points, and there is little sense in providing a drain at a high point.

There are many examples to blocked drainage system as seen in following Figure 3.1.

Figure 3.1 : Blocked Primary Drainage.

Over the last century, there has been a trend in construction towards stronger materials. By using high strength materials such as steel, more efficient, long span roofs made of smaller, shallower and more slender building elements have been possible to construct [9]. This trend is outlined in the efficiency provided by open web steel joists: very slender elements made of strong but ductile materials can lead to very efficient but very flexible structural units. All of these properties serve to make it increasingly flexible, which can increase the probability of ponding loads. While they allow for more efficient design, high strength materials and flexible roofs require careful attention to design in order to prevent ponding.

3.3 Prevention of Ponding

It seems it would be a simple matter to ensure that a roof can be stable and strong enough to withstand these loads, yet buildings continue to collapse under ponding loads. The problem in practice is that systems that are stable under the criteria provided in the literature and in the design, specifications still experience a degree of the ponding effect. A beam that is close to the critical ratio will be subjected to an amplification of the loads it experiences. A beam that is stable and strong enough to hold loads will still deflect, allowing larger loads to collect on the system.

As will be seen subsequently, this amplification factor is not accounted for in roof systems that provide a slight pitch. The two simplest ways to avoid ponding are to either increase the pitch of the whole roof, or to provide more appropriate drainage in better locations such as midspan, and conduct regular maintenance and inspection of the drainage systems. Both of these options will help to limit the water that collects on the roof and can help to prevent ponding loads. While not a cure for the problem, providing additional camber to steel joists or to the roofing system will help to reduce the effects of ponding loads. A cambered roof will gather water first at the edges, instead of at mid span, which produces much smaller bending moments and stresses in the system. This can easily be the difference between a failed and a safe roof [10].

3.4 Ponding Stability

It seems it would be a simple matter to ensure that a roof can be stable and strong enough. There are two phenomena that lead to failure under ponding loads: overload due to load amplification, and instability. While the overload condition is tested, as it is more common, stability should be also investigated. Ensuring stability of a roof system is not a simple matter, as the literature demonstrates. Many factors play a role, including the effects of two way systems, support conditions, sloped roofs, camber, and the general geometry of the system. The work done in the area has shown that ponding stability or instability can be determined, and there are various methods of doing so. Robert W. Haussler [11] initially published the most simple and widely cited ponding stability criterion in 1962, for a flat, simply supported beam, for the first time. This generally represents the worst case, and a safe way to ensure stability. It is reproduced in modified form here:

$$\pi^4 EI > \gamma\beta L^4 \tag{3.1}$$

Where E is the modulus of elasticity, I is the moment of inertia, L is the length, B is the spacing between beams and γ is the density of the fluid causing ponding on the roof. It is worth noting that the ponding problem is purely geometric. In general, the stability of a system will depend on the properties of the members and their layout in the system. The properties that determine stability are internal to the system and do not include external factors, such as the initial load. He also assumed that the ponding fluid was not held by any wall, but only rises to the level of the supports. Using this as an initial point, he found out that a stable system under water loads it can represented with the following stability requirement. If a roof was flat, provided with adequate drainage, and met this following equation, and then it would be safe from ponding loads. He also stated that any roof built on an adequate slope could not experience ponding loads, as water would simply run off.

$$\frac{\pi^4 EI}{L^4} > \gamma \tag{3.2}$$

The following years Bohannan and Kuenzi superimposed analysis of ponding loads on existing load cases by assuming linear elastic behavior and a sinusoidal deflected shape. Using energy methods, the authors determined that the work done by the load will be less than the energy in the beam where a is the length and k is the unit weight of the fluid times beam spacing.

$$\frac{\pi^4 EI}{a^4} > k \tag{3.3}$$

They concluded that if the inequality was not satisfied then the work done by load will be greater than the bending energy, and the beam is unstable. They confirmed the work of Haussler. The authors continued, however, to expand the work to the case of an original distributed load in addition to the ponding load due to the deflection. The midspan deflection resulting from both loads can be calculated as:

$$\frac{a^4 5 w_0}{384EI\left(1 - \frac{ka^4}{EI\pi^4}\right)} = \Delta \tag{3.4}$$

where w_0 is the initial uniform distributed load and all other variables are as defined above. Note that this equation is simply a combination of the critical ponding criteria and the deflection due to a uniform distributed load. It is also virtuous to notice that as a system approaches the limits for stability as defined in equations 3.4 this expression goes to infinity, and that the ponding effect amplifies the deflection due to initial loads by the factor.

4. BUILDING CODE REVIEW

According to Lawson, there are several building code provisions that do place some responsibility for rain load design on the stuructural engineer.

4.1 International Building Code (IBC) 2006

The building code, which is used in most parts of the United States, has relevant provisions for rain and ponding loads. The IBC [12] requires that roofs equipped with controlled drainage provisions shall be equipped with a secondary drainage system at a higher elevation, which prevents ponding on the roof above the design water depth. Such roofs shall be designed to sustain all rainwater loads on them to the elevation of the secondary drainage system, and the load caused by the depth of water (i.e., head) needed to cause the water to flow out of the secondary drainage system. The flow capacity of secondary (overflow) drains or scuppers shall not be less than that of the primary drains or scuppers. Ponding instability shall be considered in this situation. Moreover, for guidance on these calculations, the IBC refers designers to section 8.4 of ASCE 7.

4.2 American Society of Civil Engineers (ASCE) 7- 05

ASCE 7-05 provides information collected by experts in the field of structural engineering, as well as for structural designers. These guidelines require that two independent drainage systems to be provided, each with the same capacity. It also requires that design of a roof system has to provide adequate strength to hold standing water to the height it would reach if the primary system failed. For stability against ponding, section 8.4 requires either a sufficient slope at least 1/4, or investigation to ensure adequate stiffness against progressive deflection. It is suggested that the larger of the snow and the rain load to be used, and that the primary drainage system should be assumed to be blocked for this investigation.

4.3 American Institue of Steel Construction, 13th Edition

AISC specification Section B3.8 requires that the ponding problem should be considered in three ways. Firstly, designer needs to ensure ponding stability by either providing adequate slope at least 1/4 or adequate drainage, or the ponding investigation should performed as outlined in appendix 2 in the AISC [13] Steel Construction Manual 13th Edition.

Appendix 2 in the AISC Steel Construction Manual 13th Edition, there are two independent methods, a simplified method, which is conservative, and a more in depth method, which is more accurate.

The simplified method is developed by Burgett, and allows for a factor of safety of for against instability [14]. When using this method for trusses and joists, it is required that the moment of inertia should be reduced by fifteen percent to find the effective moment of inertia. This modification accounts for the part of deflections due to shear deflection, as opposed to that due to bending moment alone. In addition, within this method, steel decking is considered as a secondary member when it is supported directly by the primary members alone. The more accurate analysis developed by Marino [15].

4.4 AISC Design Guide 3 for Serviceability

AISC has published thirty design guides up to now in addition to the Steel Construction Manual. Design Guide 3, Serviceability Design Considerations for Steel Buildings 2nd edition contains information relevant to ponding loads. It provides a good summary of what is contained in the building codes and the AISC Appendix 2. The AISC Specification gives limits on framing stiffness that provide a stable roof system as shown in the following equations where I_d is the moment of inertia of the steel deck, I_s is the moment of inertia of secondary members, I_p is the moment of inertia of primary members, S is the spacing of secondary members, L_s is the length of secondary members, L_P is the length of primary members.

$$C_P + 0.9C_s \leq 0.25 \tag{4.1}$$

$$I_d \geq 25S^4 10^{-6};\ \ C_P \geq \frac{504 L_s\, L_P{}^4}{I_p}\ ;\ C_P \geq \frac{504 S L_s{}^4}{I_s} \tag{4.2}$$

4.5 International Plumbing Code

According to the International Plumbing Code, each portion of a roof shall be designed to sustain the load of rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow. The design rainfall shallbe based on the 100-year hourly rainfall rate or on other rainfall rates determined from approved local weather data. The following equation is the formula for the calculation of nominal rain load, R. This equation is used in section 8.4 of ASCE 7, too.

$$R = 0.0098(d_s + d_h) \tag{4.3}$$

where R is the rain load on the undeflected roof, in kN/ m^2, $\boldsymbol{d_h}$ is the additional depth of water on the undeflected roof above the inlet of secondary drainage system at its design flow, $\boldsymbol{d_s}$ is the depth of water on the undeflected roof up to the inlet of secondary drainage system when the primary drainage system is blocked. $\boldsymbol{d_h}$ and $\boldsymbol{d_s}$ are shown schematically in Figure 4.1.

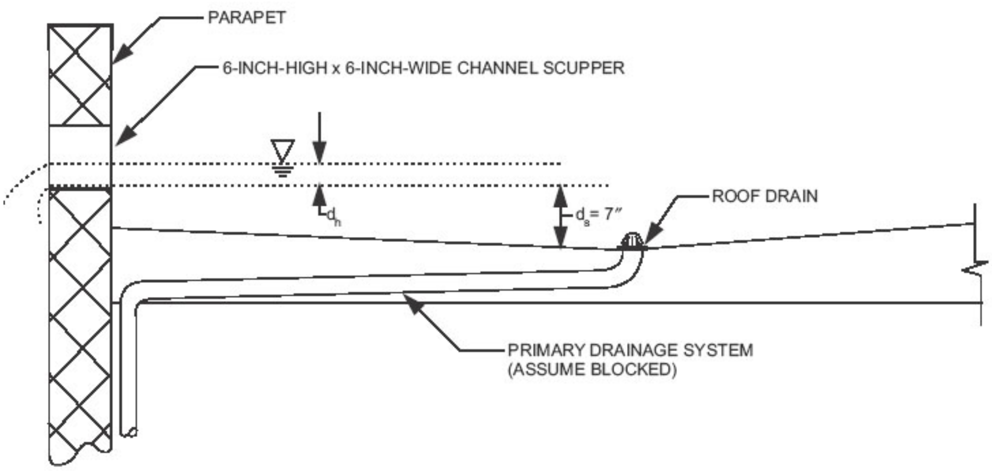


Figure 4.1 : Schematic View for d_h and d_s [16].

The nominal rain load, R, represents the weight of accumulated rainwater, assuming a blockage of the primary roof drainage system. The design of the roof drainage systems must be in accordance with Chapter 11 of the International Plumbing Code (IPC). The primary roof drainage system can include roof drains, leaders, conductors and horizontal storm drains within the structure. Drainage system design is based on specified design rainfall intensity, as well as the roof area it drains. The criteria for sizing the components of the drainage system are provided in Chapter 11 of the IPC. Where the building is configured such that water will not collect on the roof there is no requirement for a secondary drainage system as seen in Figure 4.2. Likewise, there would be no rain load required in the design of the roof.

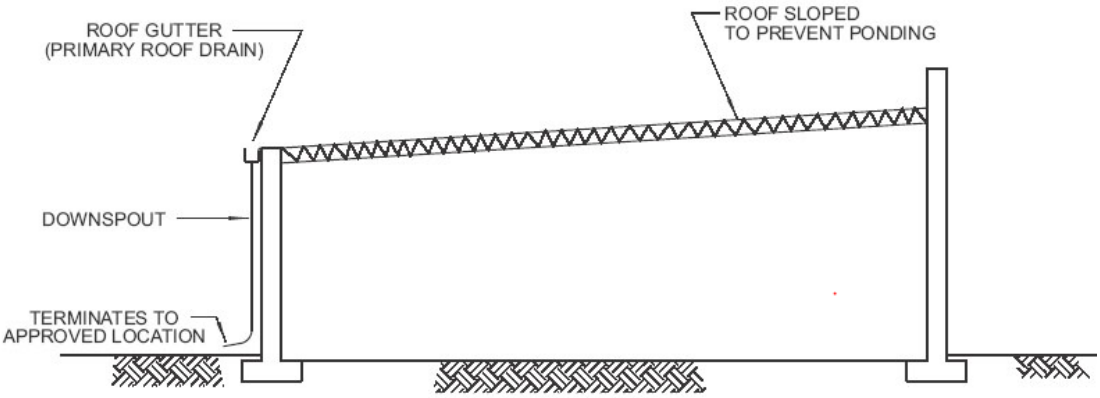


Figure 4.2 : Secondary Roof Drainage Not Required [16].

It is not rare to find that roof drains have become blocked by debris, leading to ponding of rainwater where the roof construction is conducive to retaining water. While the objective of providing roof drainage is typically to prevent the accumulation of water, the code also recognizes controlled drainage systems that are engineered to retain rainwater. The important point is that wherever the potential exists for the accumulation of rainwater on a roof, whether it is intentional or otherwise, the roof must be designed for this load. Furthermore, the IPC requires the maximum depth of water to be determined, assuming all primary roof drainage to be blocked. The water will rise above the primary roof drain until it reaches the elevation of the roof edge, scuppers or another serviceable drain. At the design rainfall intensity, this depth will be based on the flow rate of the secondary drainage system. This depth, referred to as the hydraulic head, can be determined from the previously approved data for various types of drains and flow rates.

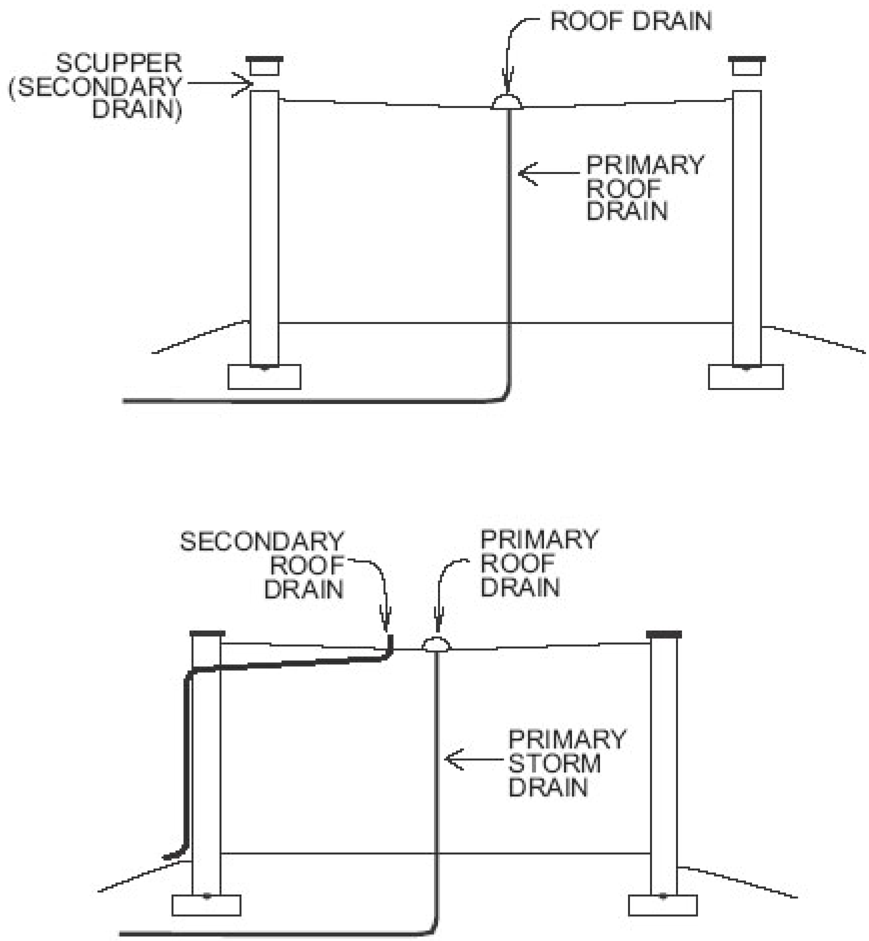


Figure 4.3 : Separate Primary and Secondary Roof Drains [16].

The IPC specifically requires a secondary roof drainage system where the building construction extends above the roof at the perimeter. This applies to parapet walls, stepped buildings or any other construction that would allow rainwater to pond on the roof. The sizing of a secondary drainage system is identical to the process used for the primary system. Instead of using a "piped" secondary system, designers may prefer to install scuppers to allow rainwater to overflow the roof. Examples of both types of secondary systems are shown in Figure 4.3. Also note that the IPC requires a secondary system to be completely separate and to discharge above grade. Since the secondary system serves as an emergency backup, requiring it to discharge above grade provides a means of signaling that there is a blockage of the primary drainage system.

4.6 Turkish Design and Construction Rules of Steel Structures

Ponding Instability has just become a subject for Turkish Design and Construction Rules of Steel Structures. In Appendix A of the Turkish Standard, ponding effect is calculated in the same manner with the American Steel Construction Manual.

To sum up, this Building Code Review session is a complete description of what building codes and design specifications requires as far as ponding loads are considered. Of course, there are many other design specifications exist which are not considered in the content of this study, but they do not have any requirements for ponding. To give an example, The National Design Specification for Wood Construction, the Building Code Requirements and the Specifications for Masonry Structures, and the Building Code Requirements for Structural Concrete are some of them. Their common point is not requiring ponding analysis or investigation. For this reason, in some cases, steel construction may be more vulnerable to ponding loads.

It should make sense that the most useful general ponding investigation procedures appear in the Steel Construction Manual and International Plumbing Code. For a structural engineer interested in ponding loads, the single section of the code that must be known is Appendix 2 of the AISC Specifications. Both the simplified method and the improved method are good ways to ensure stability, and can be easily taken direct from the literature.

5. NUMERICAL ANALYSIS

5.1 Iterative Loading

To better understand how the accumulation of water affects the stability of the double layer grid truss sytem , the trapezoidal roof structure which has length, width and depth 36m, 36m, and and 2.5m, respectively is modeled by using Sap2000 v17.3 software. Each member is 3m in length; the roof slope is about % 5. The roof consists of 365 node points and 1352 frame members. Pipe section members are selected according to the Eurocode 3-2005, by considering with ductility class high moment resisting frame system (DCG-MRF).

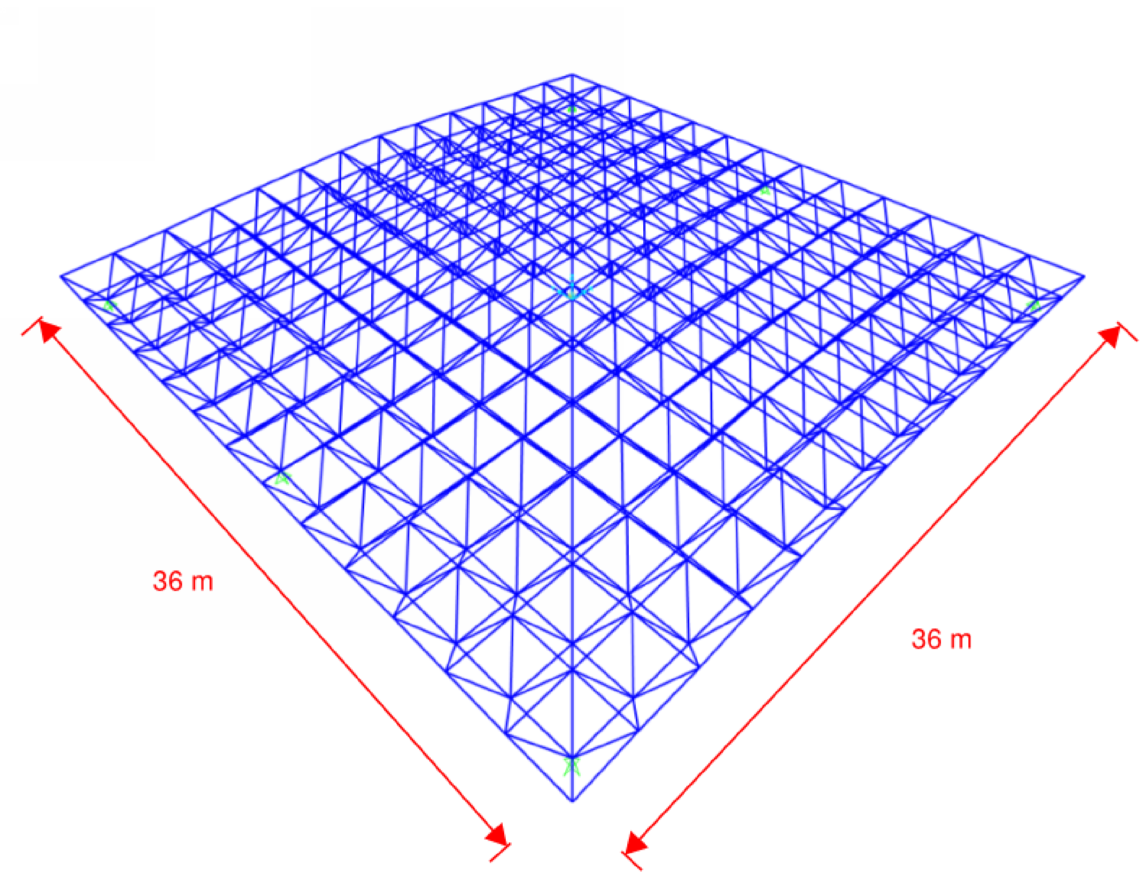
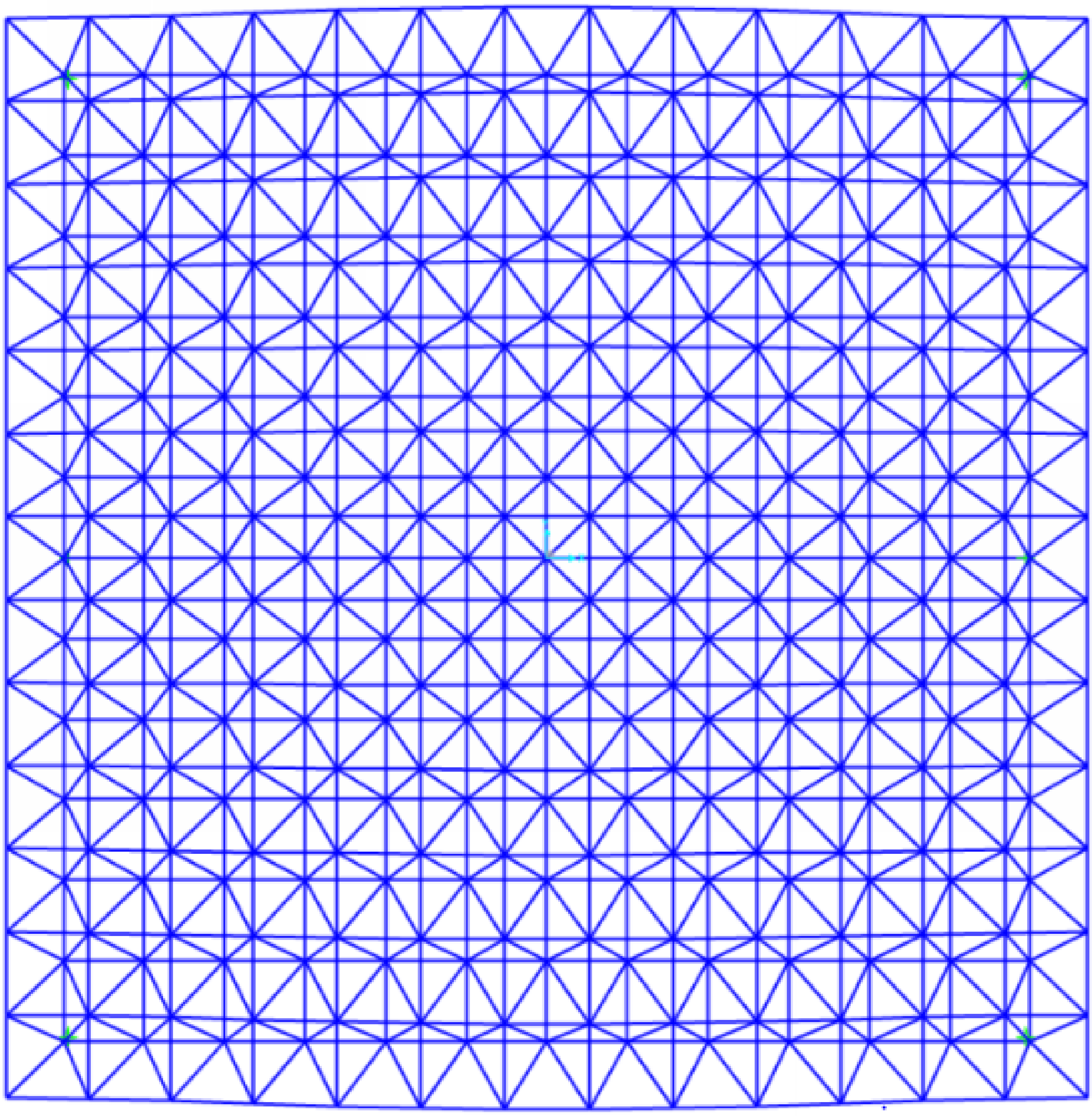


Figure 5.1 : Schematic View of 3D.



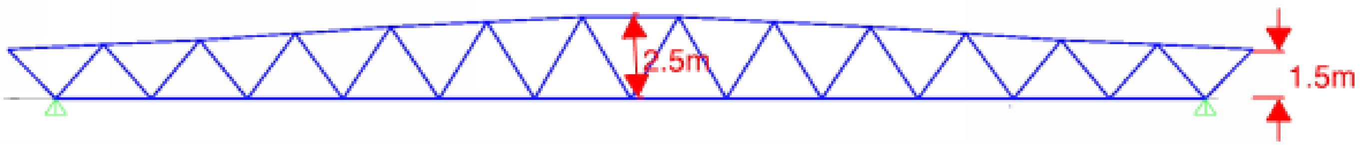


Figure 5.2 : Schematic View of Plan and Elevation.

It is assumed that somehow, primary drainage is blocked and water raises as time goes on. There is 1m diffference between the highest and lowest points of the roof joints. Firstly, the roof joint is loaded to 5 cm height to see how the water raises affects the roof stability. Every linear static loading increased by 5 cm height, and repeated 20 times up to 100 cm in height.

Load combinations are created as:

1,35D + 1,5L(for strength);

1,35D + 1,5L + 1, 5W_5(for strength); 1D + 1L + 1W_5(for deflection)

1,35D + 1,5L + 1, 5W_10(for strength); 1D + 1L + 1W_10(for deflection)

1,35D + 1,5L + 1, 5W_15(for strength); 1D + 1L + 1W_15(for deflection)

1,35D + 1,5L + 1, 5W_20(for strength); 1D + 1L + 1W_20(for deflection)

1,35D + 1,5L + 1, 5W_25(for strength); 1D + 1L + 1W_25(for deflection)

1,35D + 1,5L + 1, 5W_30(for strength); 1D + 1L + 1W_30(for deflection)

1,35D + 1,5L + 1, 5W_35(for strength); 1D + 1L + 1W_35(for deflection)

1,35D + 1,5L + 1, 5W_40(for strength); 1D + 1L + 1W_40(for deflection)

1,35D + 1,5L + 1, 5W_45(for strength); 1D + 1L + 1W_45(for deflection)

1,35D + 1,5L + 1, 5W_50(for strength); 1D + 1L + 1W_50(for deflection)

1,35D + 1,5L + 1, 5W_55(for strength); 1D + 1L + 1W_55(for deflection)

1,35D + 1,5L + 1, 5W_60(for strength); 1D + 1L + 1W_60(for deflection)

1,35D + 1,5L + 1, 5W_65(for strength); 1D + 1L + 1W_65(for deflection)

1,35D + 1,5L + 1, 5W_70(for strength); 1D + 1L + 1W_70(for deflection)

1,35D + 1,5L + 1, 5W_75(for strength); 1D + 1L + 1W_75(for deflection)

1,35D + 1,5L + 1, 5W_80(for strength); 1D + 1L + 1W_80(for deflection)

1,35D + 1,5L + 1, 5W_85(for strength); 1D + 1L + 1W_85(for deflection)

1,35D + 1,5L + 1, 5W_90(for strength); 1D + 1L + 1W_90(for deflection)

1,35D + 1,5L + 1, 5W_95(for strength); 1D + 1L + 1W_95(for deflection)

1,35D + 1,5L + 1, 5W_100(for strength); 1D + 1L + 1W_100(for deflection)

By the way, 0, 0225 kN/ m^2 the coating weight is taken as the live load.

Every 9 m square are thaught as one-way slab and the load caused by the weight of the accumulated water is assigned to joints. Changing depth is calculated for every 5 cm of incrementation of water by using Autocad 2013 software program.

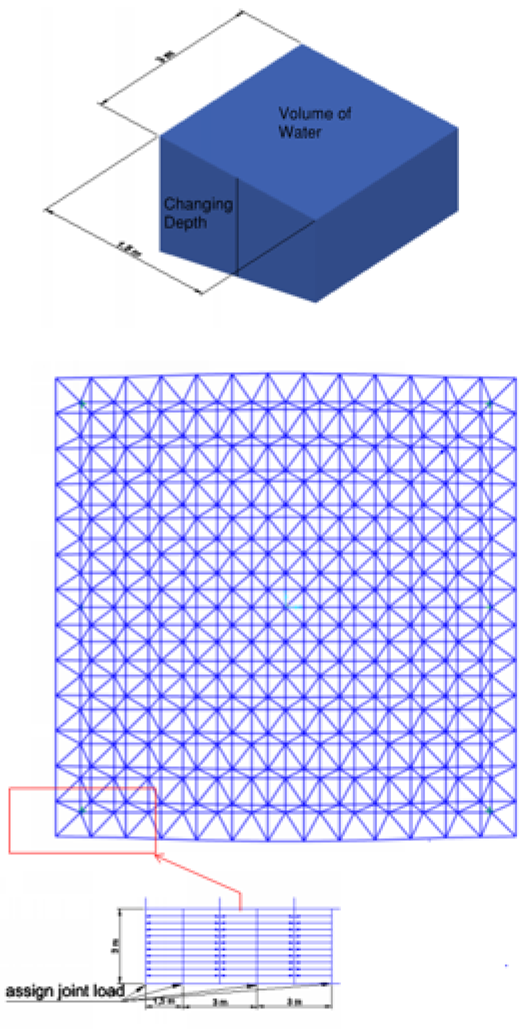


Figure 5.3 : Ponding Load Combination and Application.

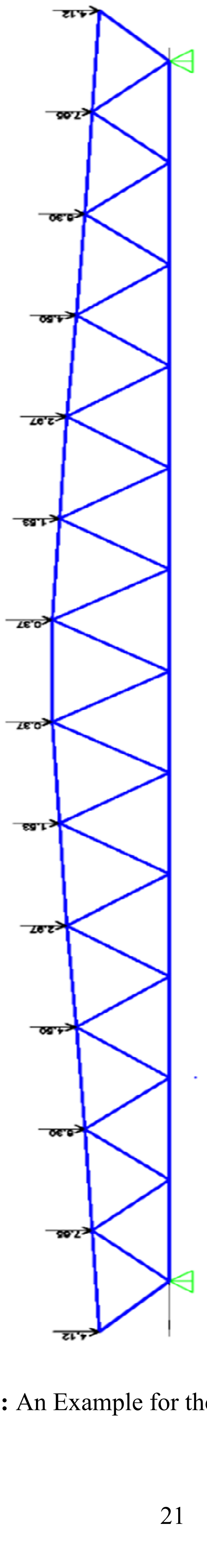


Figure 5.4 : An Example for the Loading at 100 cm.

Every single loading is analzyed according to the Eurocode3 Section 6.3.3 (4) which part explains about the uniform members in bending and axial compression.

Table 5.1 : Eurocode 3 Section 6.3.3 (4).

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \qquad (6.61)$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \qquad (6.62)$$

where N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$ are the design values of the compression force and the maximum moments about the y-y and z-z axis along the member, respectively

$\Delta M_{y,Ed}$, $\Delta M_{z,Ed}$ are the moments due to the shift of the centroidal axis according to 6.2.9.3 for class 4 sections, see Table 6.7,

χ_y and χ_z are the reduction factors due to flexural buckling from 6.3.1

χ_{LT} is the reduction factor due to lateral torsional buckling from 6.3.2

k_{yy}, k_{yz}, k_{zy}, k_{zz} are the interaction factors

Table 6.7: Values for $N_{Rk} = f_y A_i$, $M_{i,Rk} = f_y W_i$ and $\Delta M_{i,Ed}$

Class	1	2	3	4
A_i	A	A	A	A_{eff}
W_y	$W_{pl,y}$	$W_{pl,y}$	$W_{el,y}$	$W_{eff,y}$
W_z	$W_{pl,z}$	$W_{pl,z}$	$W_{el,z}$	$W_{eff,z}$
$\Delta M_{y,Ed}$	0	0	0	$e_{N,y} N_{Ed}$
$\Delta M_{z,Ed}$	0	0	0	$e_{N,z} N_{Ed}$

NOTE For members not susceptible to torsional deformation χ_{LT} would be $\chi_{LT} = 1,0$.

The interaction factors k_{yy}, k_{yz}, k_{zy}, k_{zz} depend on the method which is chosen.

NOTE 1 The interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} have been derived from two alternative approaches. Values of these factors may be obtained from Annex A (alternative method 1) or from Annex B (alternative method 2).

NOTE 2 The National Annex may give a choice from alternative method 1 or alternative method 2.

NOTE 3 For simplicity verifications may be performed in the elastic range only.

According to Eurocode3 Section 6.3.3 (4) interaction factors can be derived from the following tables;

Table 5.2 : Eurocode3 Section 6.3.3 (4) Table A.1 and B.1 Interaction Factors.

Table A.1: Interaction factors k_{ij} (6.3.3(4))

Interaction factors	Design assumptions: elastic cross-sectional properties class 3, class 4	Design assumptions: plastic cross-sectional properties class 1, class 2
k_{yy}	$C_{my} C_{mLT} \dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{my} C_{mLT} \dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}} \dfrac{1}{C_{yy}}$
k_{yz}	$C_{mz} \dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz} \dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}} \dfrac{1}{C_{yz}} 0,6 \sqrt{\dfrac{w_z}{w_y}}$
k_{zy}	$C_{my} C_{mLT} \dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{my} C_{mLT} \dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,y}}} \dfrac{1}{C_{zy}} 0,6 \sqrt{\dfrac{w_y}{w_z}}$
k_{zz}	$C_{mz} \dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz} \dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,z}}} \dfrac{1}{C_{zz}}$

Table B.1: Interaction factors k_{ij} for members not susceptible to torsional deformations

Interaction factors	Type of sections	Design assumptions: elastic cross-sectional properties class 3, class 4	Design assumptions: plastic cross-sectional properties class 1, class 2
k_{yy}	I-sections RHS-sections	$C_{my}\left(1+0,6\bar{\lambda}_y \dfrac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) \le C_{my}\left(1+0,6\dfrac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$	$C_{my}\left(1+\left(\bar{\lambda}_y-0,2\right)\dfrac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right) \le C_{my}\left(1+0,8\dfrac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$
k_{yz}	I-sections RHS-sections	k_{zz}	$0,6\ k_{zz}$
k_{zy}	I-sections RHS-sections	$0,8\ k_{yy}$	$0,6\ k_{yy}$
k_{zz}	I-sections	$C_{mz}\left(1+0,6\bar{\lambda}_z \dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right) \le C_{mz}\left(1+0,6\dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$	$C_{mz}\left(1+\left(2\bar{\lambda}_z-0,6\right)\dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right) \le C_{mz}\left(1+1,4\dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
	RHS-sections		$C_{mz}\left(1+\left(\bar{\lambda}_z-0,2\right)\dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right) \le C_{mz}\left(1+0,8\dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
For I- and H-sections and rectangular hollow sections under axial compression and uniaxial bending $M_{y,Ed}$ the coefficient k_{zy} may be $k_{zy} = 0$.			

During the first loadings at height 5cm and 10 cm, there is no member failure has been observed. After the thirdiest loading, number of members failed is increased drastically. The following Figure 5.5 shows the number of failed members.

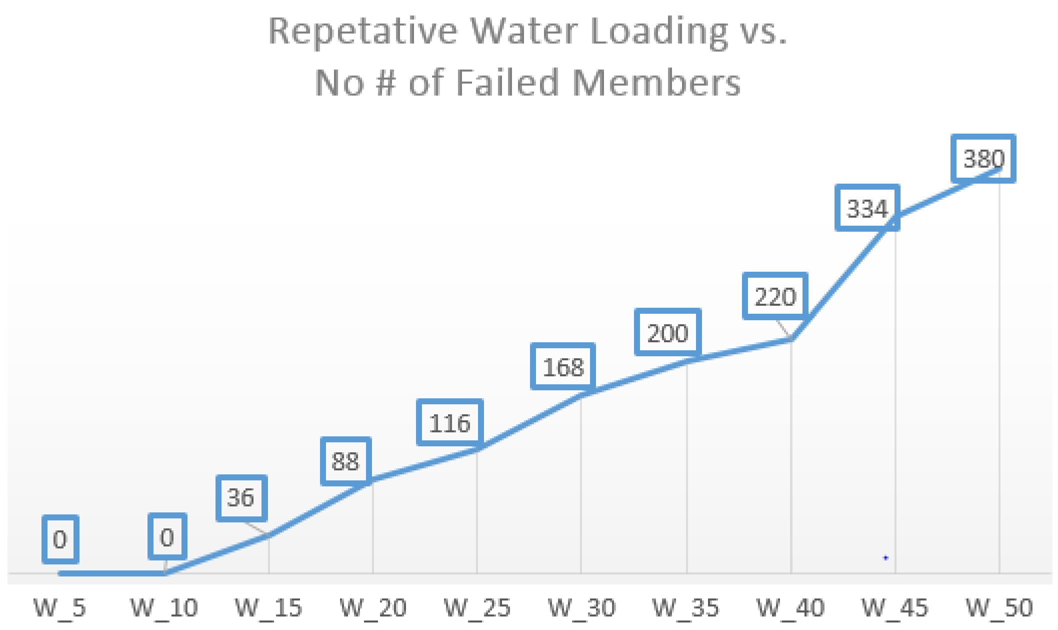


Figure 5.5 : Repetative Loading vs. No # of Failed Members.

The literature uses the stress or strain limit as a truss structure failure criterion. That is, if the strain or stress in a certain member exceeds the limit value of the material fracture happens, and this member can not support the stress or strain any longer.

Compression is considered to be one reason that element failures can result. The relationship between stress and strain is linear until the member axial tensile or compressive stress reaches the material's yield value, $\sigma_{crit.}$ [17].

Then, the material behavior is proposed to be plastic until failure happens. However, the exact critical strain value at the point at which failure happens is not quite clear. After failure occurs, the fracture member may have one or two free ends. If the external force is not applied, unloading will occur in the failure member. The unloading is elastic in nature with the same slope as the initial loading phase, also presented in the following Figure 5.6 and Figure 5.7.

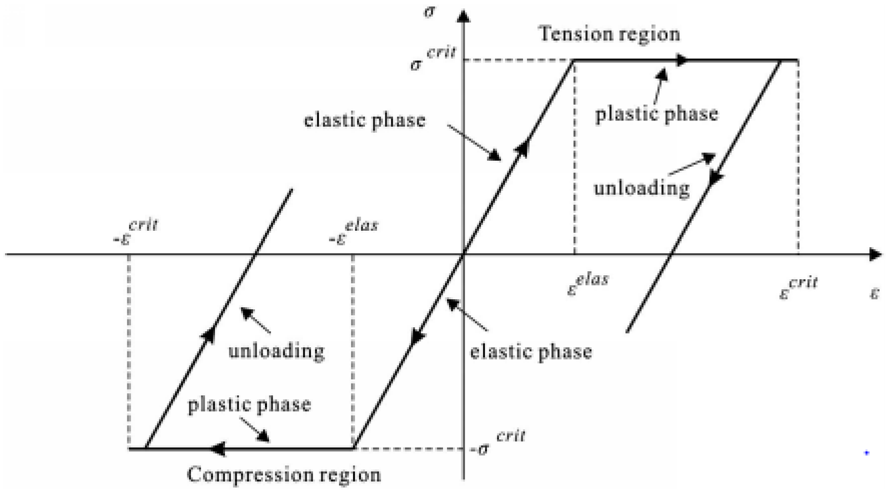


Figure 5.6 : Ideal-Plastic Stress-Strain Curve for Failure Criterion.

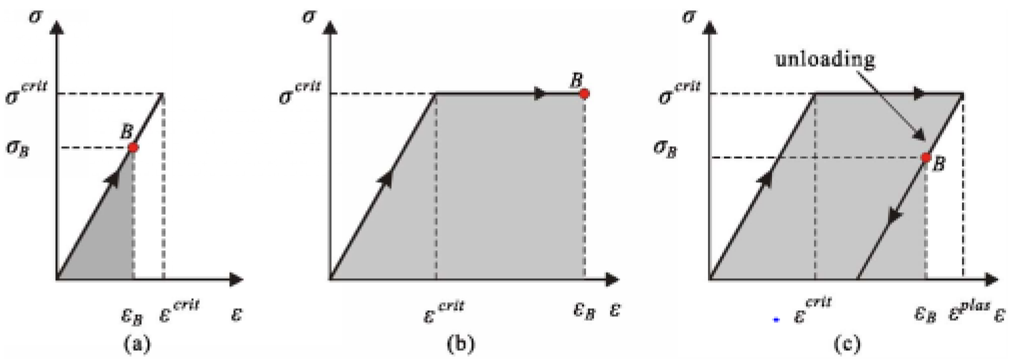


Figure 5.7 : Strain Energy Phases: (a)Elastic; (b)Plastic; (c)Unloading.

Let' s look at the schematic view of Sap2000 model of the roof structure to criticize the number and the type of members failed.

In the following pages, the members in red are failed according to Eurocode3 Section 6.3.3 (4);

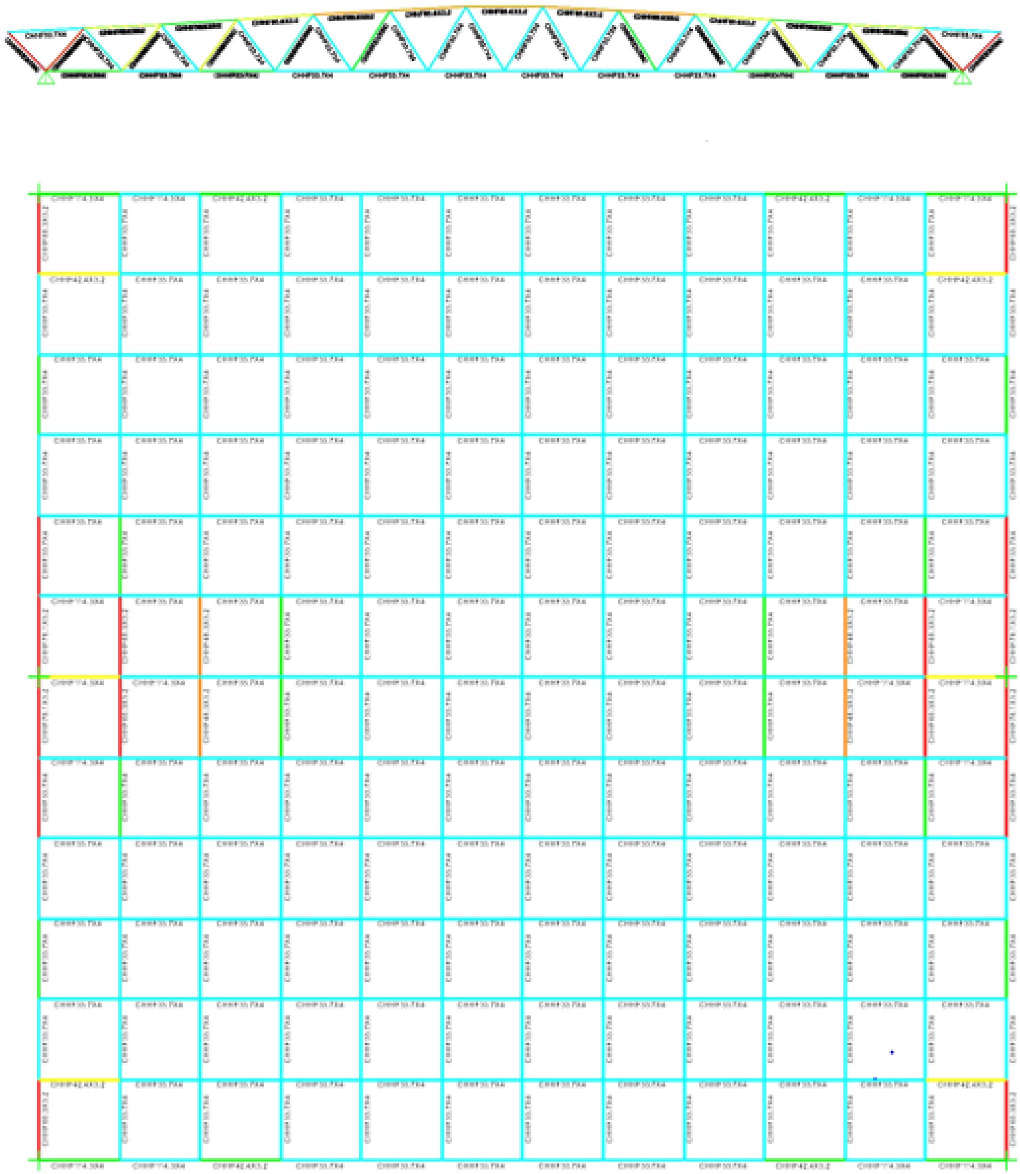


Figure 5.8 : Loading at height 15 cm.

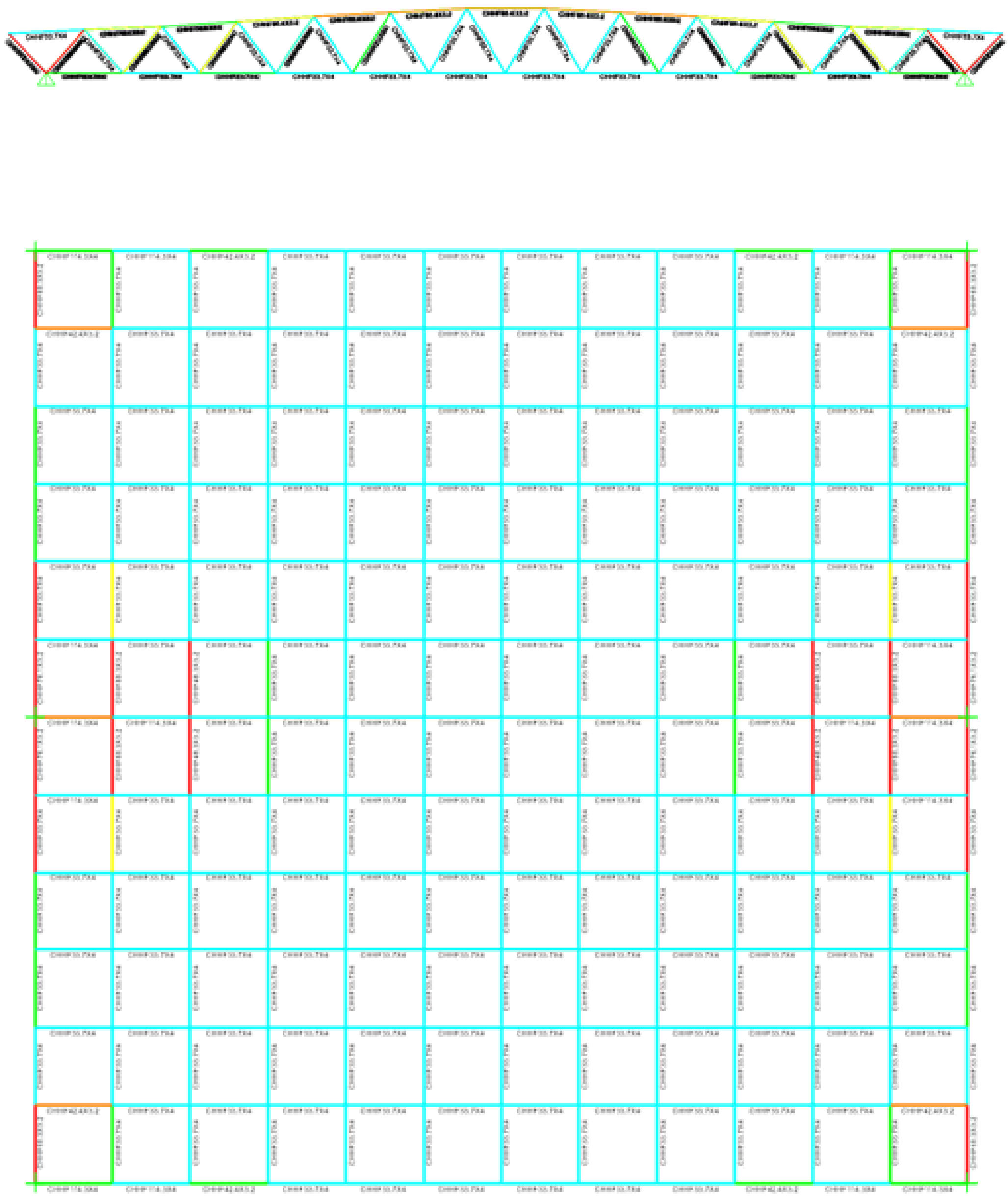


Figure 5.9 : Loading at height 20 cm.

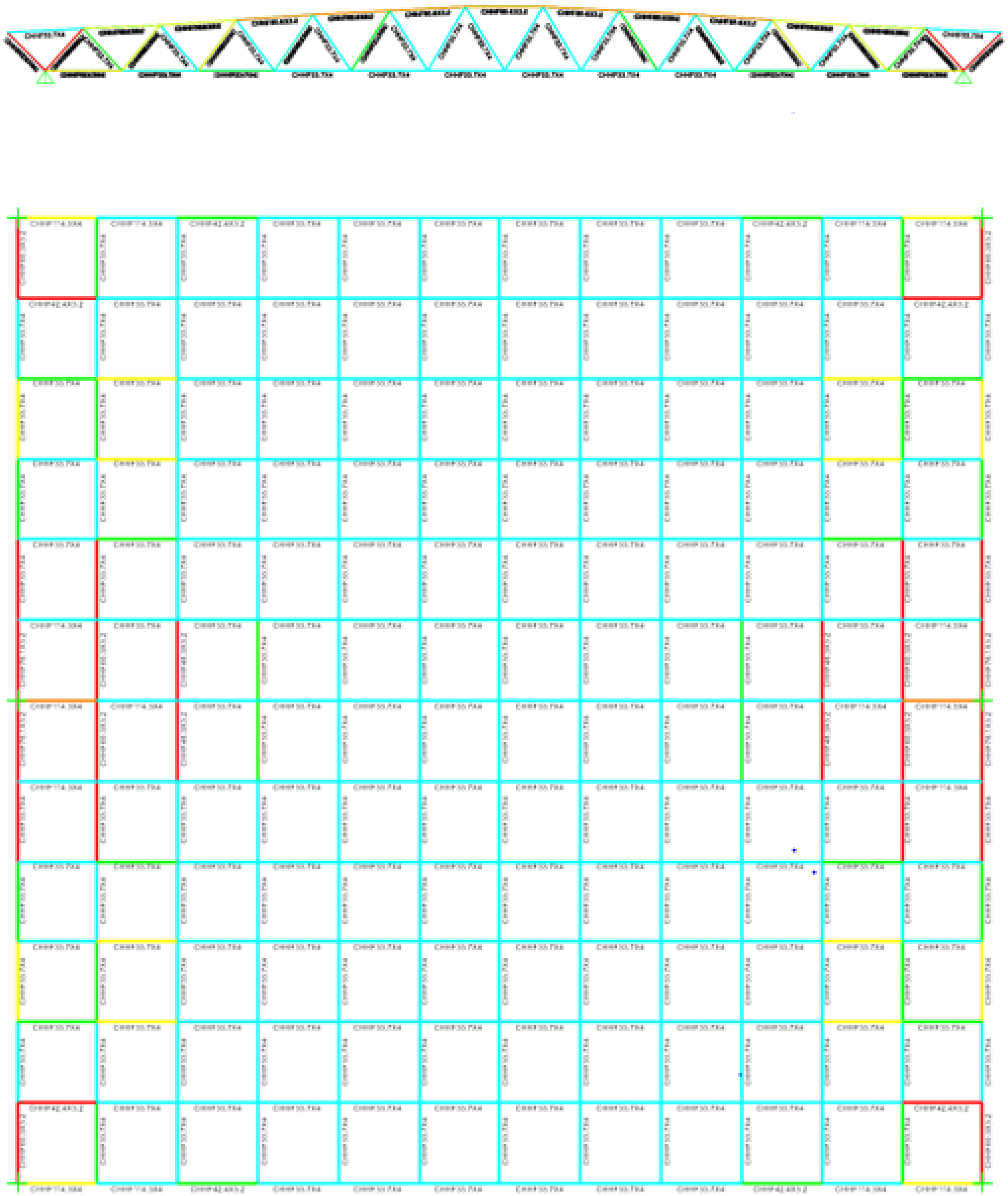


Figure 5.10 : Loading at height 25 cm.

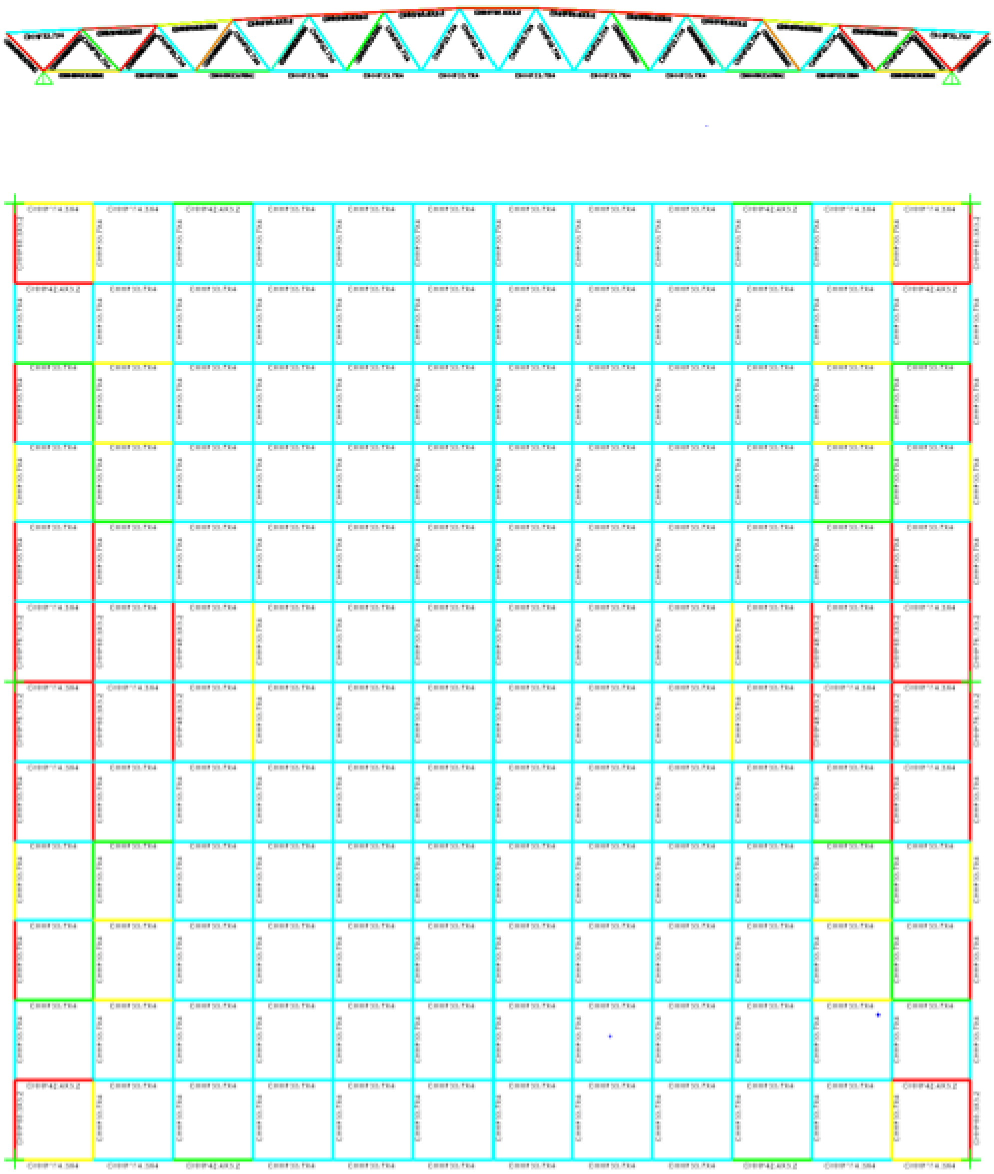


Figure 5.11 : Loading at height 30 cm.

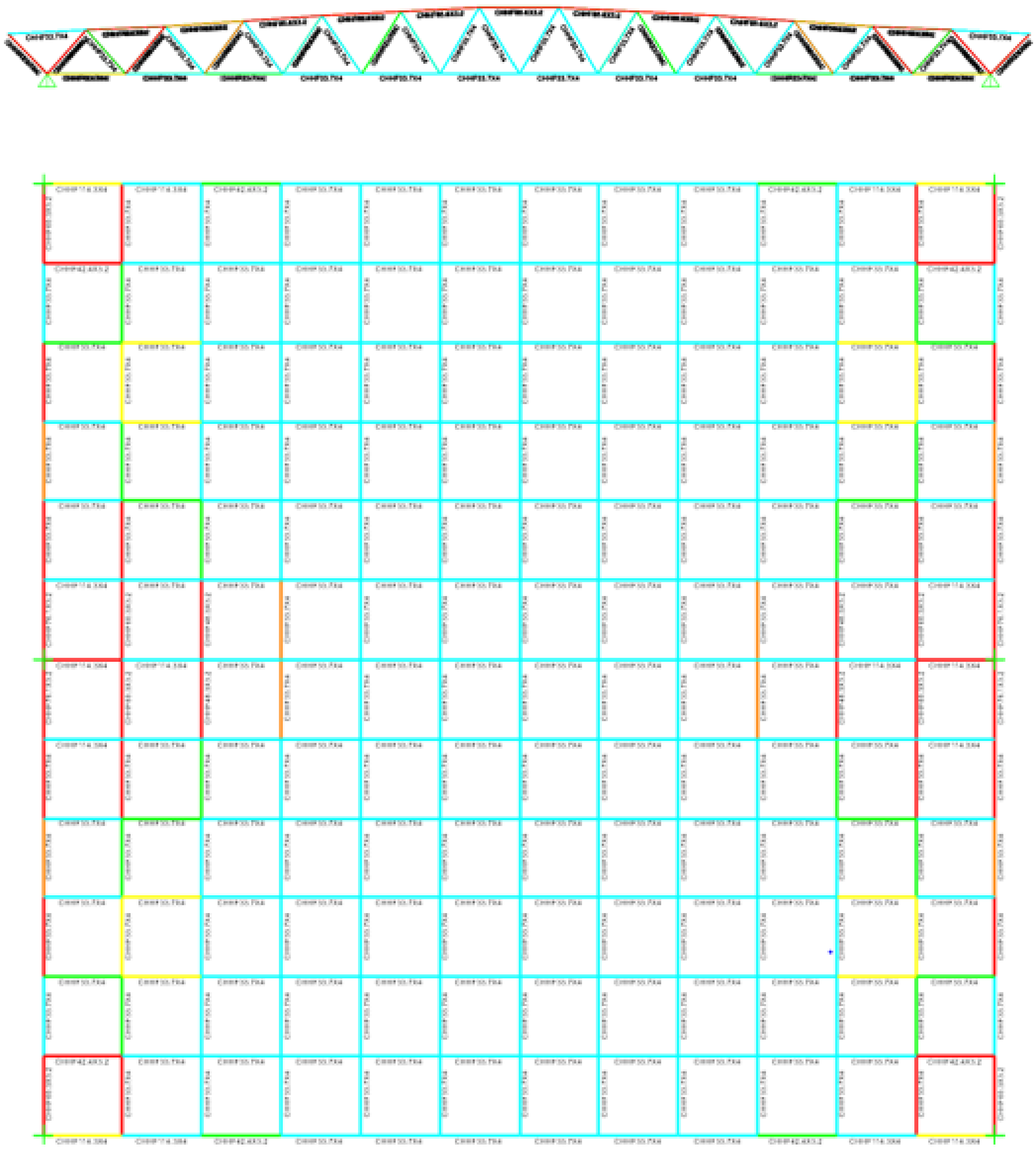


Figure 5.12 : Loading at height 35 cm.

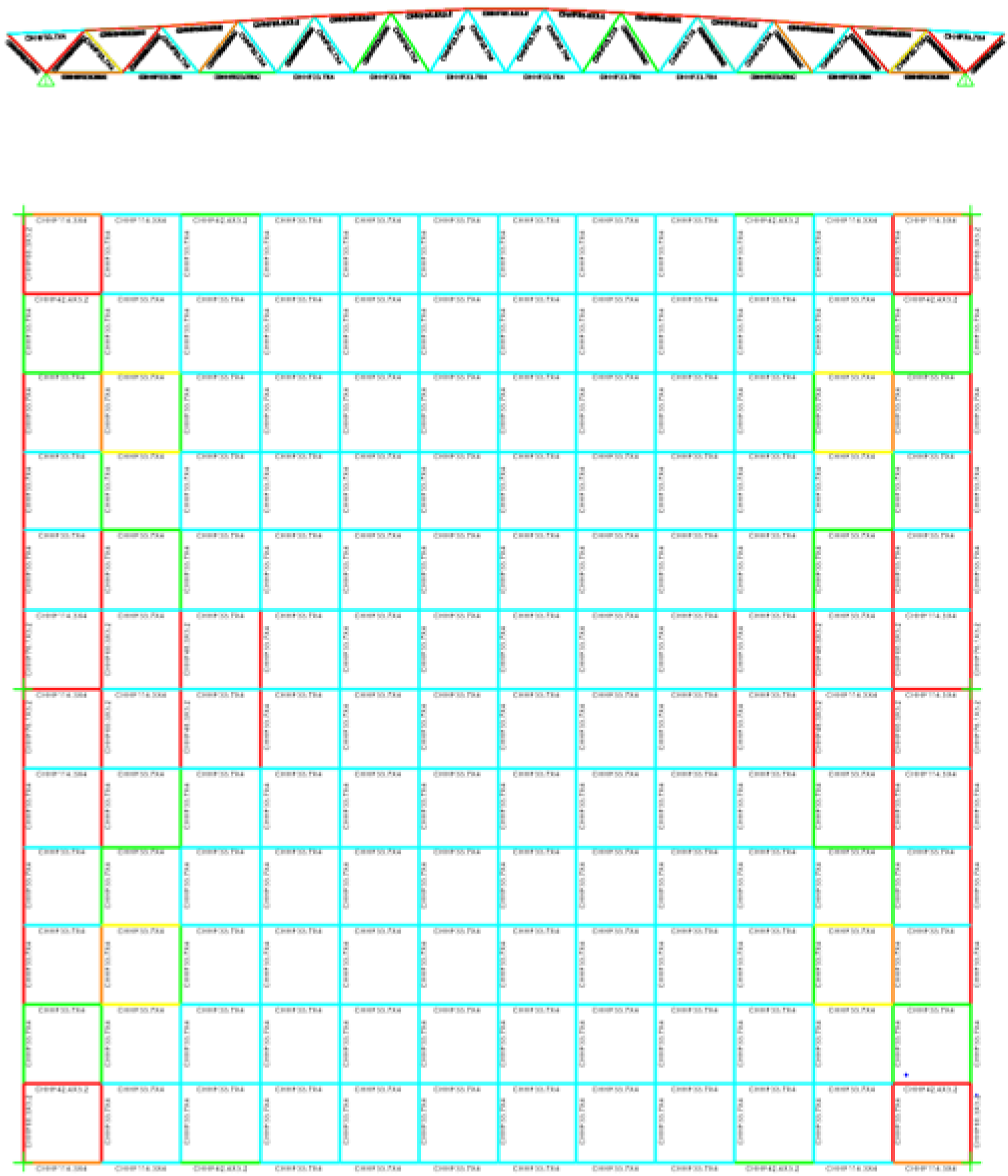


Figure 5.13 : Loading at height 40 cm.

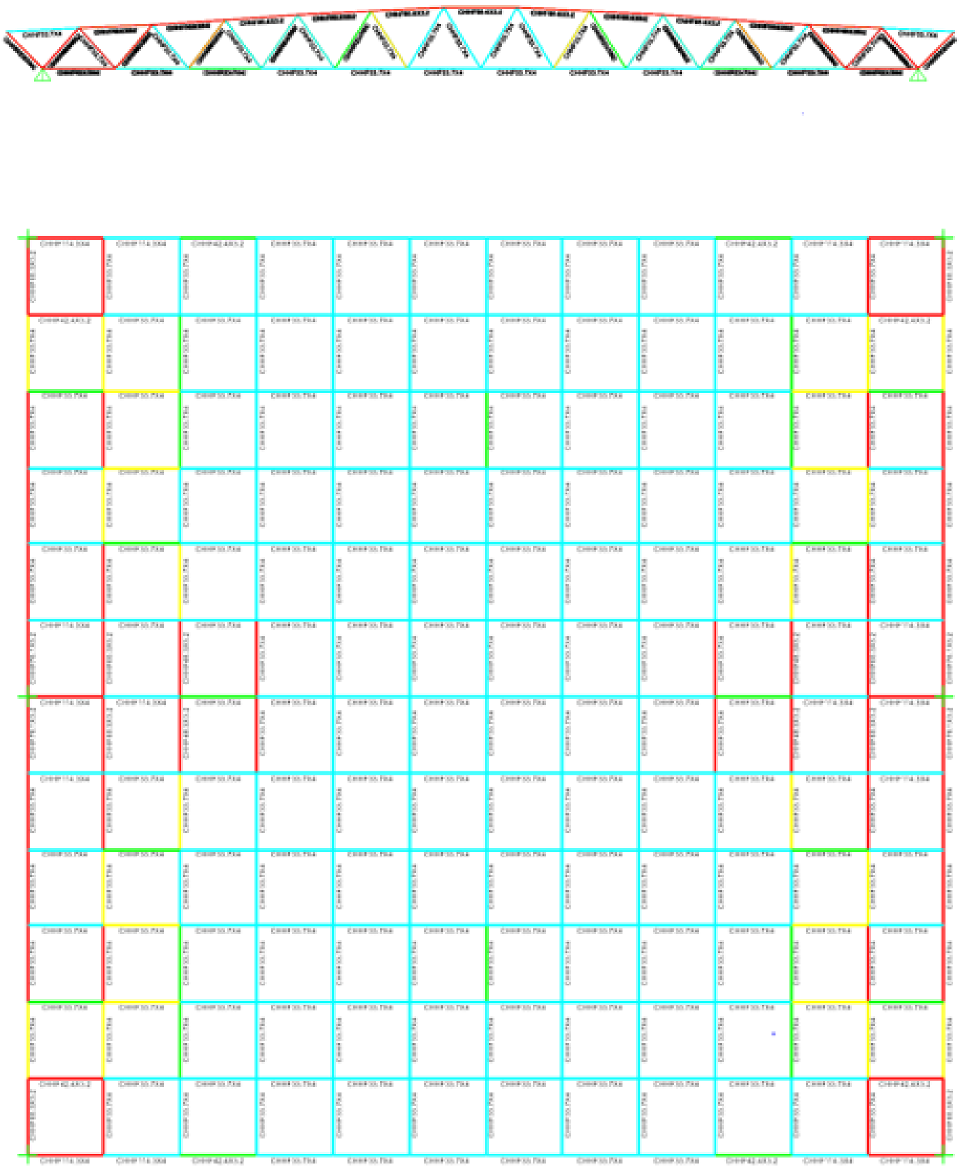


Figure 5.14 : Loading at height 45 cm.

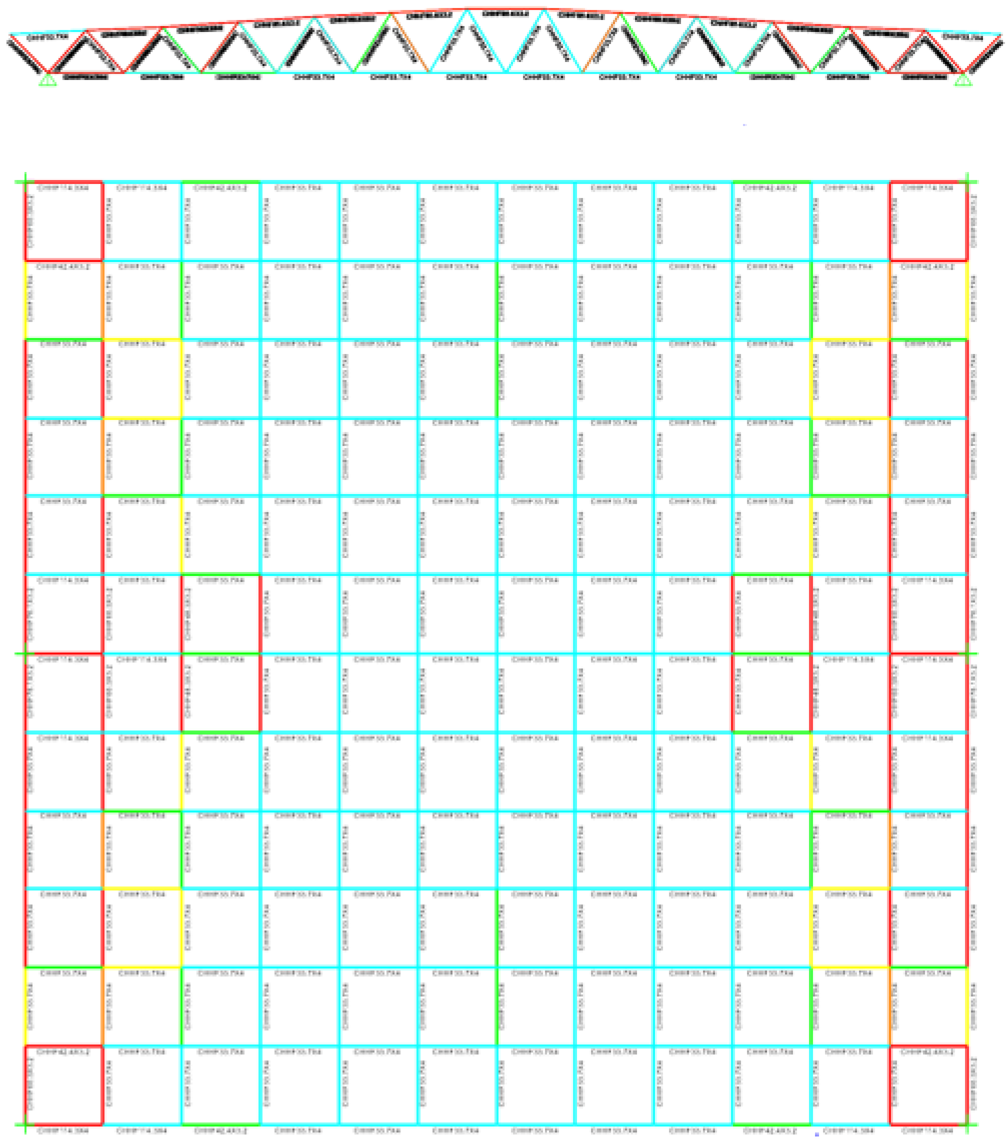


Figure 5.15 : Loading at height 50 cm.

As stated before, the loading is repeated until the height of 100 cm but the schematic views are not shown here.

In the following analyzes, the failed members will be removed from the model and the model is analyzed in that way.

The following Figure 5.16 satisfies that most of the members failed are located at the edge side of the roof structure since the water increases as time goes on.

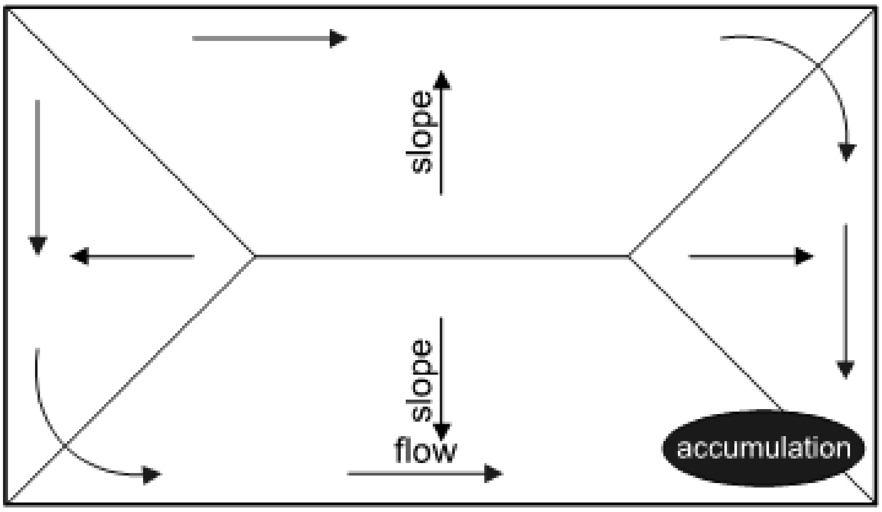


Figure 5.16 : Schematic View of Accumulated Water.

5.2 Iterative Loading and Members Failed

In the previous section, the load is applied increasingly until the height of 100 cm but the failed members remained in the structure. It means eventhough members are failed due to bending, they remained in the structure. Now, the failed members will deleted from the model and analysis will be done, repeatly. Firstly, members failed in the thirdiest loading, which gives failures at height 15 cm, are grouped and deleted. The new model has saved with the new name after deleting members. And loading at height 20 cm is applied to the new model, and analysis is repeated. The following Figure 5.17 shows the deleted members.

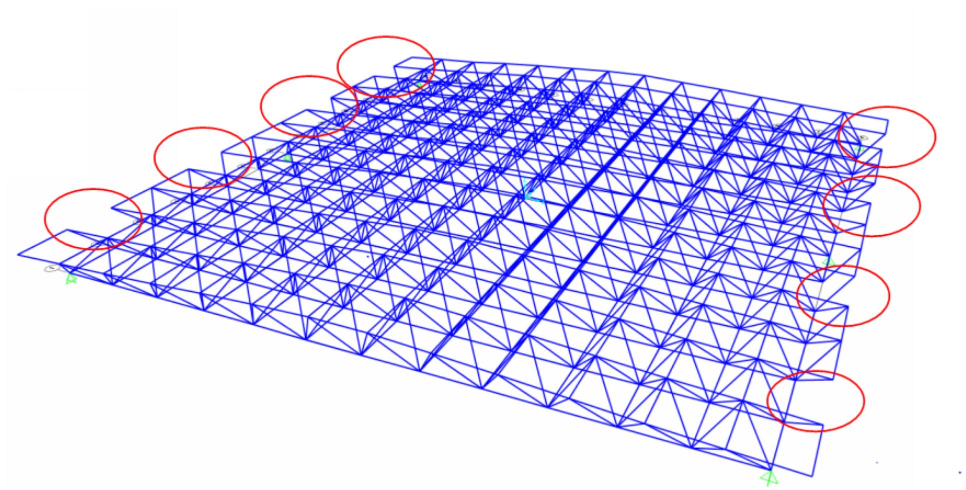


Figure 5.17 : Places of Deleted Members.

The following Figure 5.18 shows the members failed at the edge of the roof due to deflection.

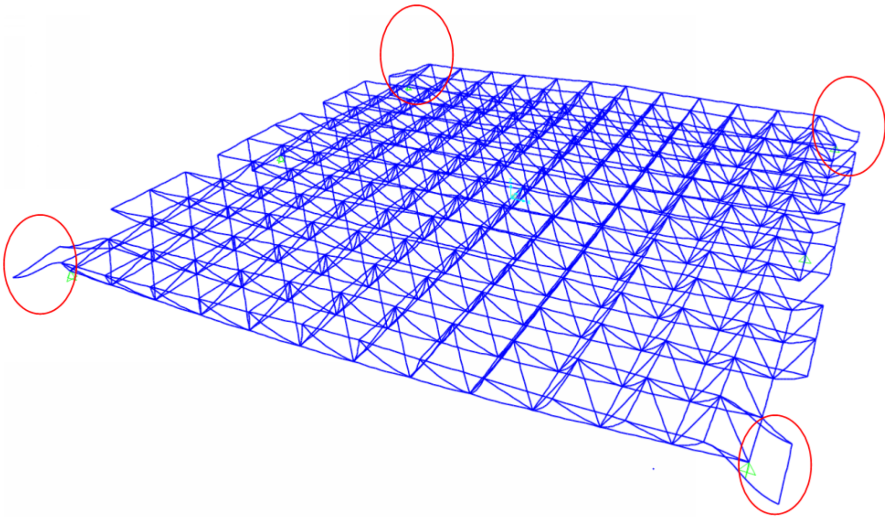


Figure 5.18 : Failed Members due to Deflection.

In flat or drainage blocked roof systems, beams or joist may have some initial sag or deflection allowing water to pool or collect, causing more deflection, and thus more load to collect, and thus more deflection, and so forth. This progressive deflection and loading sequence of events may lead to a ponding instability failure where the water weight eventually overcomes the roof structure strength. Sufficient roof slope and/ or roof stiffness is necessary to prevent ponding instability. While members become deflected their potential for pooling water increases, as well.

The following Figure 5.19 shows the potential for pooling water of deflected member. Anexpression can be established to calculate the slope θ from curvature anywhere along a bending member of constant modulus of elasticity E and moment of inertia I.

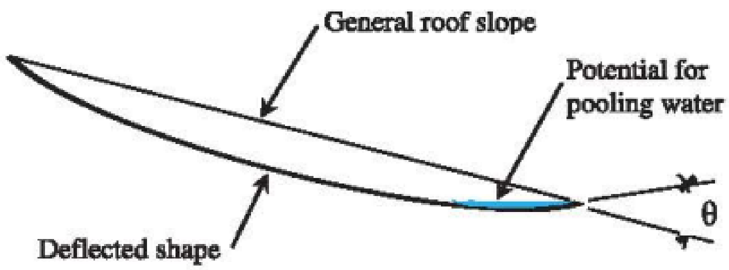


Figure 5.19 : The potential for Pooling Water of Deflected Member.

The following Figure 5.20 shows failed members which is about 210 in quantity and has colour in red.

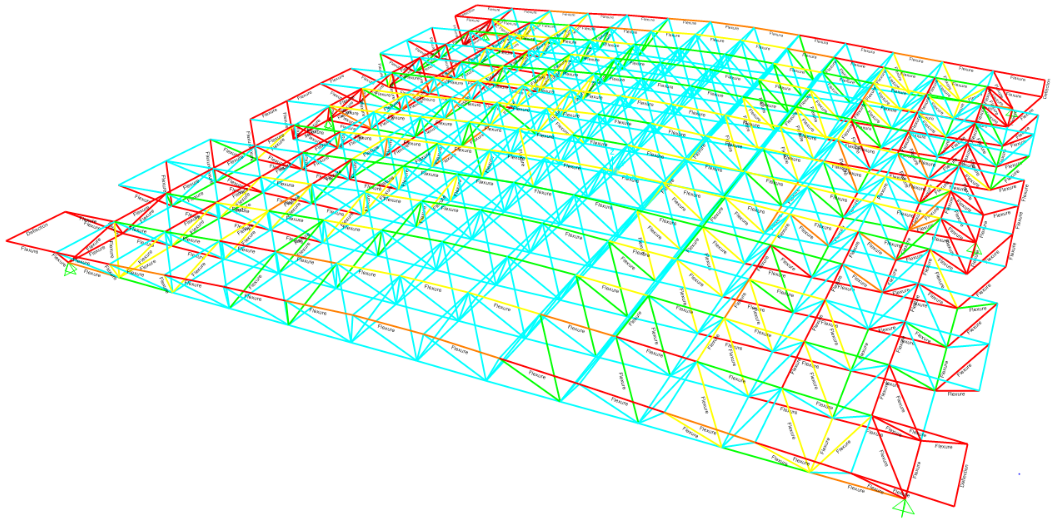


Figure 5.20 : Failed Members.

The following Figure 5.21 shows the places of deleted members after loading at height 20 cm.

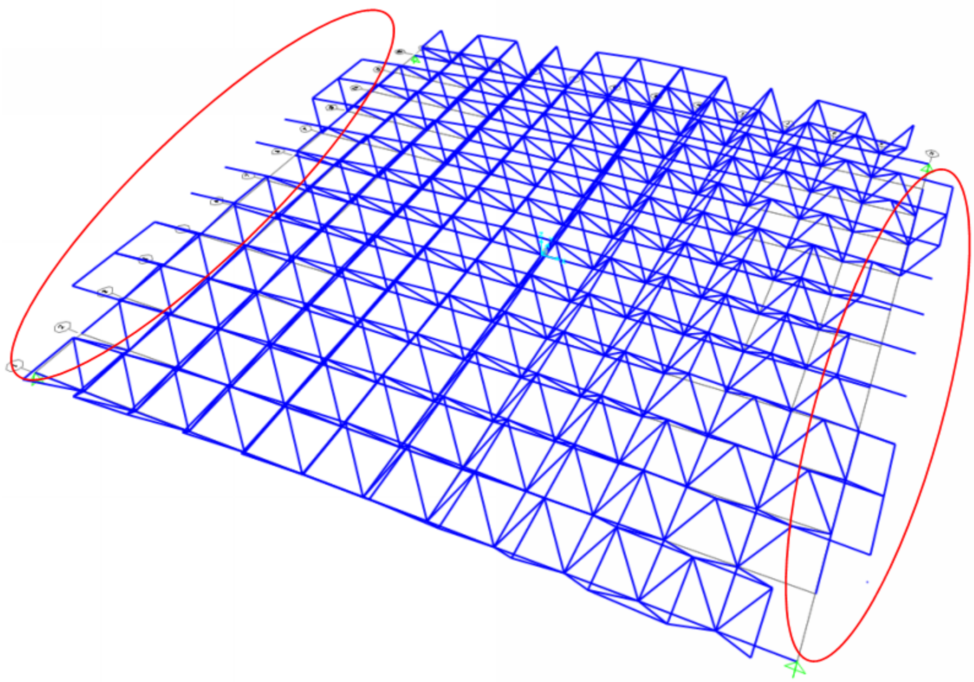


Figure 5.21 : Deleted Members.

After deleting the failed members and saving the model with a new name, loading at height 25 cm is applied to the new model and analysis is repeated.The failed members can be seen in the following Figure 5.22.

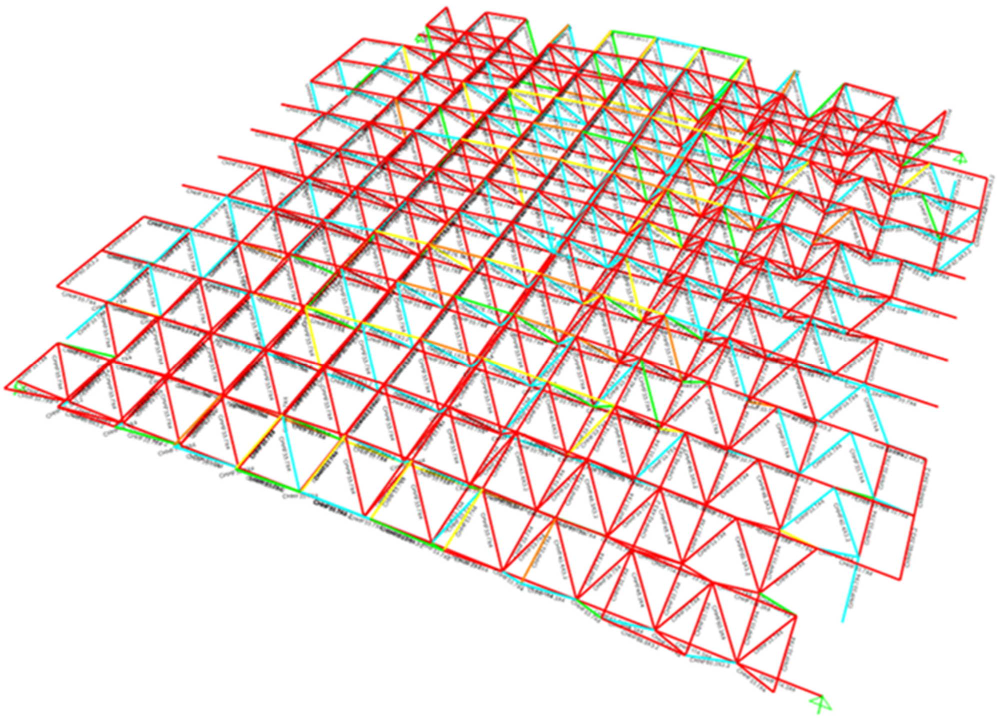


Figure 5.22 : Loading at height 25 cm.

Then the number of failed member is 774, while looking the failed members it is observed that most of the members failed due to flexure are bottom chord members and diagonals, and due to deflection are top chord members, only.

Since deflection limit is taken as L/240 for combination of dead and live load, the deflection exceeds that limits, too. The roof structure becomes unstable due to deflection and buckled members.

In the first group of iterative loading, the failed members have not been deleted from the model. While comparing these two group of loadings; in the first group even loading at height 50 cm there has been 380 members failed, in the second group loading at 25 cm there has been 774 members failed. It can be resulted that because of their large degree of static indeterminancy, space trusses are often assumed to have sufficient redundancy such that the loss of one member would cause force redistributions that can be accommodated by the remaining structure [18].

The collapse mechanics in double layer space truss roof member is rather diffucult to analyze with linear systems and it requires progressive collapse analyses. Because the force redistributions may cause members to exhibit nonlinear behavior and yield in the case of a tension member or buckle in the case of a compression member [19].

However, because of strain hardening, a yielded tension member can typically absorb additional force, whereas a compression member resists decreasing force for increasing shortening after reaching its buckling force. Thus, a compression member cannot resist additional force but has to shed force and cause additional force redistributions into other members. These other members might also buckle and cause further force redistributions, and, thus, failure can progress through the structure to cause collapse.

This collapse mechanism is exacerbated when the buckling member has a rapid rate of decrease in post-buckling force capacity and the truss module adjacent to the member is very compliant. In such situations, two equilibrium positions and forces between the post-buckling member and its adjacent truss module are possible. Thus the member will snap-through to a low post-buckling load, typically 40% of the buckling load. The snap-through phenomenon described elsewhere [20], [21] and [22] causes major redistributions, which often cause failure in nearby members and lead to progressive collapse throughout the structure with no increase in external load. In addition, because the snap-through phenomenon is rapid, dynamic effects can increase the force redistribution further. However, dynamic effects can also cause material properties to differ, e.g., by increasing the yield strength, so tending to offset the larger forces due to dynamic redistribution.

In other words, [23] when the snap-through which is the nonlinear instability region occurs then the equilibrium path goes from one stable point 1 to another new stable point 2. The nonlinear behavior places the critical limit load at point 1 equal to that at point 2, but the load limit corresponds to a new structural shape. The second stable point along the equilibrium path occurs after a large displacement of the structure. During this snap-through behaviour, the slope of the equilibrium path (load versus deflection) finally becomes zero. The slope of this curve is also referred to as the "tangent stiffness". When the tangent stiffness moderates and approaches zero for a single-degree-of freedom system, many nonlinear solvers come across convergence problems. Some solvers immediately jump to point 2 without identifying the unstable snap-through path. Bifurcation buckling is also shown in the following Figure 5.23 with a linear, prebuckling region along the equilibrium path up to a critical load point ($P_{critical}$). At the bifurcation point the structure immediately becomes unstable and buckles. The member is unable to support any additional load, which is not the case for nonlinear snap-through buckling, and immediately becomes unstable and buckles. The member is unable to support any additional load, which is not the case for nonlinear snap through buckling.

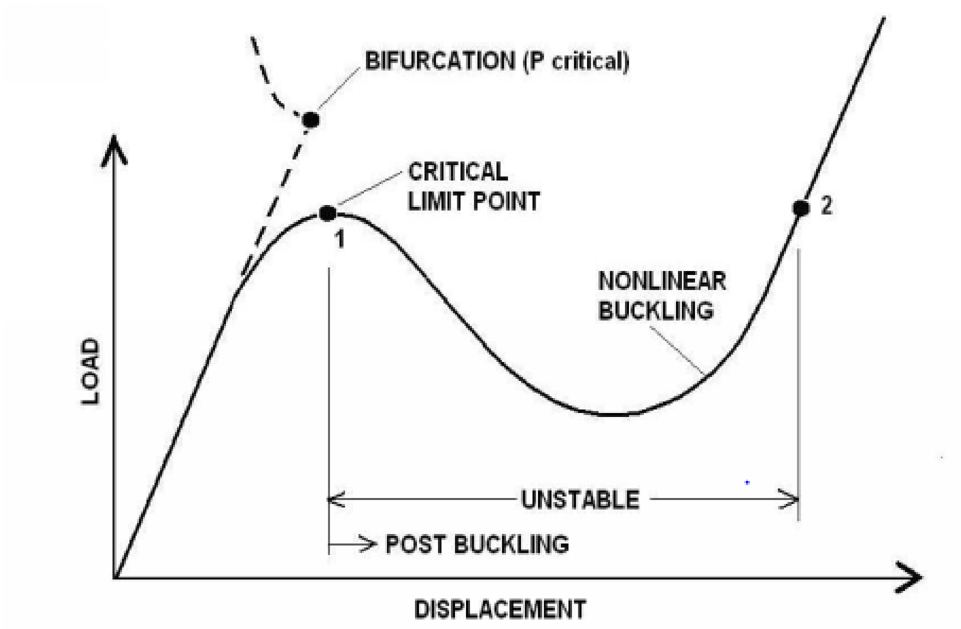


Figure 5.23 : Equilibrium Paths for Nonlinear Bifurcation Buckling.

5.3 Progressive Collapse Mechanism

Two approaches are used for providing resistance to progressive collapse, namely, the indirect method or the direct methods [24].

The indirect method is a prescriptive approach of providing a minimum level of connectivity between various structural components, and little additional structural analysis is required by the designer. In general, in place of calculations demonstrating the effects of abnormal loads on buildings, the designer may use an implicit design approach that incorporates measures to increase the overall strength of the structure. The direct methods, on the other hand, rely heavily on structural analysis. The designer explicitly considers the ability of the structure to resist the effects of an unusual load event such as ponding load. Analysis of a building's resistance to progressive collapse can be performed using various methods, ranging from linear elastic static, inelastic static analysis to sophisticated non-linear elastic and inelastic dynamic finite element analysis. The physics involved in the spread of a localized failure in a structure can be complex. Therefore, the designer should be careful about the tools that are used to analyze a given structure. These tools should be adequate to detention the desired information. For instance, if inertia effects are to be included and energy dissipation in the form of plastic action needs to be computed, it is imperative to perform a non-linear dynamic analysis should performed. On the other hand, if simple parametric studies are to be performed to study redistribution of loads when certain elements of a structure are removed, then simple linear elastic static analysis may be enough.

5.4 Progressive Collapse Mechanism: Alternate Path Method Analysis

Ellingwood and Leyendecker (1978), performed the alternate path method analysis in the structure at the design load with a member removed to see if the structure can tolerate the redistributed loads due to the "damage" [25]. They developed and calibrated the following criteria for use with the alternate path method in terms of the nominal resistance R' and loads.

$$\emptyset \mathrm{R}' \geq D + 0.45\, L + 0.20\, W \tag{5.1}$$

where D = dead load; L and W are loads specified by American National Standard ANSI A58.1-1982 (ANSI 1982), and $\emptyset$ is the resistance factor.

In this thesis study Ellingwood and Leyendecker' s equation is used to since it is the first time that ASCE 7/ANSI A58 introduced a requirement for progressive collapse due to "local failure caused by severe overloads" in Section 1.3.1 of ANSI Standard A58.1-1972, the first edition following the 1968 Ronan Point collapse (see Appendix A). No commentary or other guidance was provided. ANSI Standard A58.1-1982, Section 1.3, retitled General Structural Integrity, contained a more comprehensive performance statement, and referred to a greatly expanded commentary section and references for guidance. The 1988 and 1993 editions (now titled ASCE Standard 7) illustrated several structural system layouts that would lead to development of alternate load paths. Section 1.4 of ASCE 7-95 retained the performance requirement that a building will be designed to sustain local damage, with the structural system as a whole remaining stable. However, the commentary was shortened, keeping the discussion of general design approaches to general structural integrity but eliminating the figures and other specific guidance. At the same time, a new Section 2.5 was added that required a check of strength and stability of structural systems under low-probability events, where required by the authority having jurisdiction (AHJ). The provisions in ASCE 7-98 and ASCE 7-02 are essentially the same as in the 1995 edition. The (non-mandatory) Commentary C2.5 recommends the following load combination for checking the ability of a damaged structure to maintain its overall stability for a short time following an abnormal load event.

$$(0.9\ or\ 1.2)\ D\ +\ (0.5\ L\ or\ 0.2\ S)\ +\ 0.2\ W \tag{5.2}$$

in which D, L, S and W are specified as dead, live, snow and wind loads respectively determined according to Sections 3, 4, 6 and 7 of ASCE 7-02. This check suggests the notional removal of selected (presumably damaged) load-bearing elements at the discretion of the engineer without stipulating tolerable damage. If certain key elements in the structural system must be designed to withstand the effects of the accident (perhaps to allow the development of alternate load paths), they should be designed using the following combination,

$$(0.9 \text{ or } 1.2)\ \mathrm{D}\ +\ \mathrm{Ak}\ +\ (0.5 \text{ L or } 0.2 \text{ S}) \tag{5.3}$$

in which Ak is the postulated action due to the abnormal load. Normally, only the main load-bearing structure would be checked using these combinations.If linear analysis is used, then the factor of safety for each member can be determined by comparing the member capacity with the member force obtained by analysis, this can be done in the previous sections 5.1 and 5.2 while doing the iterative loading.

in which Ak is the postulated action due to the abnormal load. Normally, only the main load-bearing structure would be checked using these combinations.If linear analysis is used, then the factor of safety for each member can be determined by comparing the member capacity with the member force obtained by analysis, this can be done in the previous sections 5.1 and 5.2 while doing the iterative loading.

If all member factors of safety are greater than one, progressive collapse should not occur by neglecting the dynamic effects. However, some of the members with factors of safety less than unity might fail, and their failure would cause load redistributions, which, in turn, could cause other members to fail and lead to progressive collapse. Linear analysis can not supply this information, but it can suggest whether progressive collapse is likely, depending on the numbers of members with low factors of safety.

On the other hand, nonlinear collapse analysis can be used to determine the capacity of the system when the member is omitted. The nonlinear analysis will yield the system factor of safety, which, if less than unity, indicates progressive collapse would occur.

The alternate path methods described assume that static analysis adequately accounts for the effects of a member being damaged. If the loss of the function of a member is gradual, then the redistributions will be gradual, and, therefore, static analysis should be adequate. However, if the member loss is sudden and the load on the truss is non-zero, then dynamic effects can come into play. Locally, impact effects will be felt by neighboring members of the space truss module.

In addition, as the structure falls to a new equilibrium position, the work done that does not get redistributed as strain energy or get dissipated plastically in the members will cause the structure to pass the new equilibrium position, thereby temporarily increasing the member forces. That increase cause the failure of a member, then collapse progress. Thus static alternate path analysis is only an approximate simulation of the phenomenon. In the following sections the dynamic response of the space truss will be handled using both linear and nonlinear alternate path analysis methods. While doing this study, Murtha-Smith's (1988) methodology is examplary [18].

The Sap2000 model shown in Figure 5.24 below is created before and used for analysis again. The design was generated using repetitive elastic analyses and design modification until no member was overstressed, and no member could have a smaller section without overstressing another member.

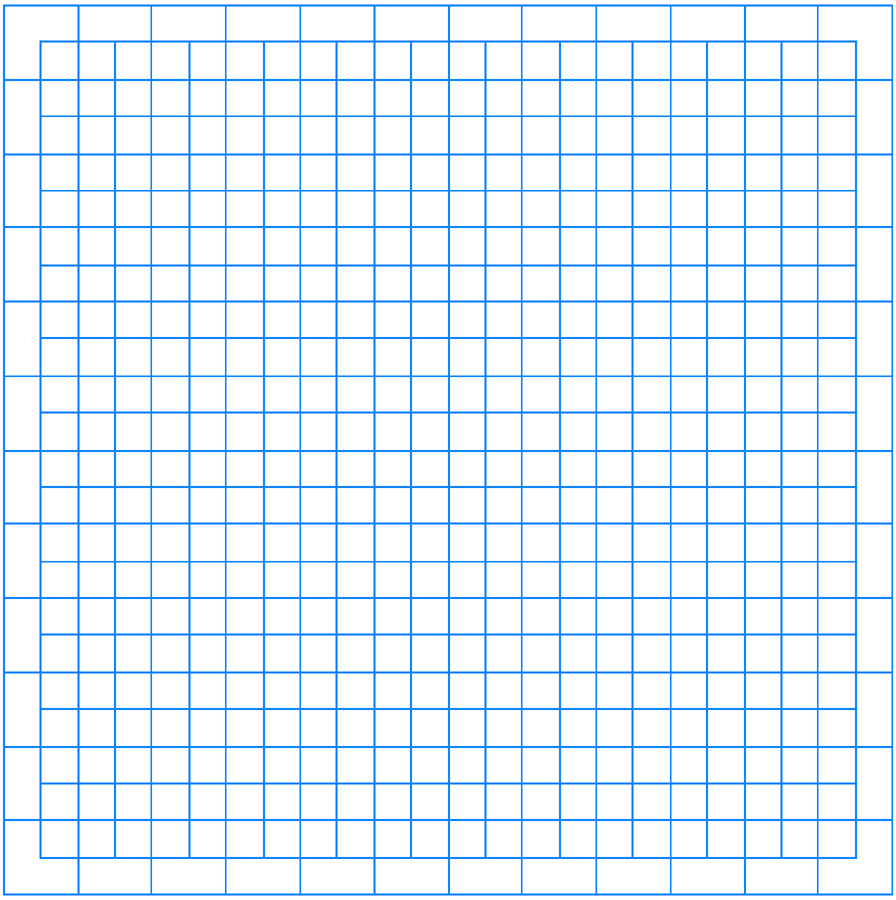


Figure 5.24 : Sap2000 Model.

5.4.1. Methodology

The 12 by 12 bay space truss was analyzed using both linear and nonlinear alternate path analysis methods. In each analysis, only one member was removed. For the linear analyses, the member factors of safety, were determined; for the nonlinear analyses, the system collapse factor of safety was determined. For the linear analyses, 28 different members (factor of safety greater than 2.5) were separately considered to be removed, whereas only one of the 28 members was considered for the nonlinear analysis.

5.4.2. Members play role on stability

If any specific case does not progress, space trusses are often designed to have approximately uniform member factors of safety. It means some members can be considered as more critical than others. Especially, chord members carrying large forces, and strain energy would cause the largest force and energy redistributions. Moreover, chord members along the column lines at the midspan and over the columns carry larger forces, thus can be critical. If, those chord members toward the center of the span along the column line are surrounded by small-size diagonal members, the failure becomes inevitable due to the loss of a chord member. In addition, the diagonals, which, near the column supports also carry larger forces and can be critical.

5.4.3. Linear alternate path analyses

The space truss was analyzed using the linear alternate path method for the distinct removal of the 28 members shown in the following Figure 5.25. In each analysis, only one of the 28 members was removed, while the other 27 remained part of the structure. Thus the analyses yielded the factors of safety in all the members due to the loss of a single member; they did not simulate the progression of collapse.

Where, U, L, D stands for Upper Chord, Lower Chord and Diagonal, respectively.

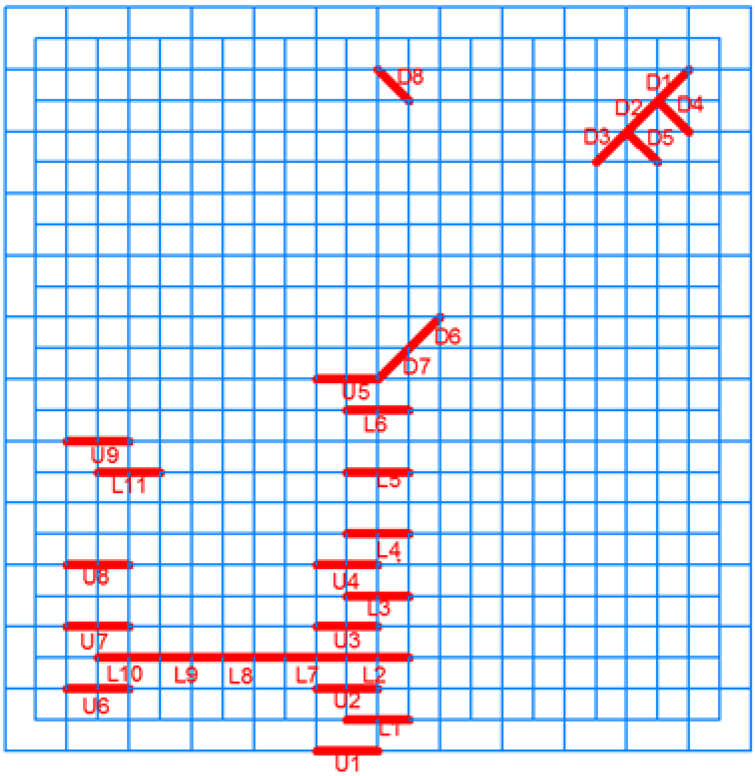


Figure 5.25 : Critical Members L1-L11,U1-U9,D1-D8, Individually Removed.

Table 5.3 shows the results of the analyses for linear alternate path analysis for full service load and equation 5.1 conditions. It indicates for each separately removed member the number of members in group A, with factors of safety less than 1.0, and in group B, with factors of safety greater than or equal to 1.0 but less than 1.25. As repeated at the beginning of section 4, the factor of safety was defined as the ratio of the member capacity to the member force.

While it is to be expected that some or all of group A will fail and that the members in group B are vulnerable to failure, particularly if members in group A fail, only a nonlinear analysis can give more detailed indications. Although all members removed initially had factors of safety approximately equal to 2.0, it can be seen by observing the results in Table 5.3 that removal of some members will cause major damage and quite probably collapse, whereas removal of other members may cause only partial damage. For example, the removal of L2 or U2 would cause 17 or 12 members, respectively, to be in group A, and 25 or 23 members, respectively,to be in group B. It is very likely that removal of L2 or U2 at full service load would lead to progressive collapse. On the other hand, removal of L5 or U5 probably would not lead to progressive collapse at the design load. Similarly, removal of diagonals D2, with 19 members in group A, compared with diagonal D3, with no members in group A, indicates that D2 is far more critical than D3 with respect to progressive collapse.

From the Table 5.3, it can be seen that loss of a chord member or diagonal leads to major losses in safety in the adjacent diagonals or chord members, respectively. In addition, loss of a chord member leads to significant losses of safety in adjacent parallel chord members. To summarize, the linear alternate path analyses indicate that critical chord members appear to be those in the middle half of the span along and adjacently parallel to the column line, and critical diagonal members appear to be those adjacent to the reaction points.

Table 5.3 : Results for Linear Alternate Paths Analysis.

Critical member		NUMBER OF MEMBERS IN GROUPS			
		Dead and Full Live Loading		(Dead + 0.45L)/Resistance factorØ	
Location	Number	A	B	A	B
Only one Lower Chord Member Removed					
Across truss centerline	L1	11	13	7	10
	L2	17	25	11	16
	L3	5	10	5	2
	L4	2	2	2	0
	L5	0	4	0	2
	L6	2	2	0	4
Along column centerline	L7	12	20	8	9
	L8	13	12	7	10
	L9	8	10	4	6
	L10	0	1	0	0
Across column centerline	L11	0	0	0	0
Only one Upper Chord Member Removed					
Across truss centerline	U1	3	8	1	2
	U2	12	23	6	15
	U3	8	13	4	4
	U4	3	4	1	4
	U5	0	0	0	0
Across column centerline	U6	4	1	4	0
	U7	4	0	15	6
	U8	0	0	0	0
	U9	0	0	0	0
Only one Diagonal Member Removed					
Adjacent to column	D1	4	4	4	0
	D2	19	8	15	6
	D3	0	0	0	0
	D4	12	9	6	7
	D5	1	1	1	0
At center of truss	D6	0	0	0	0
	D7	0	0	0	0
Across column centerline	L11	0	0	0	0
Following removal of critical member, factors of safety of members in-group A are <1.0, and in group B are a 1.0 but =1.25.					

5.4.4. Nonlinear alternate path analyses

The space truss also analyzed using the collapse analysis of space trusses (CAST) program fully described by Smith (1984) and summarized in Appendix 2. The truss was analyzed with member LI in place and then removed to determine the collapse load and mode for the two cases. The design service load was 4.07 kPa, and the undamaged truss collapse capacity given by the CAST program was 7.14 kPa, giving a system factor of safety of 1.75 by comparing the system capacity with the system design load. The analysis for the damaged truss with LI removed gave a capacity of 55% of the undamaged capacity. This gives a system factor of safety of 0.96, indicating collapse at a load less than the design service load.

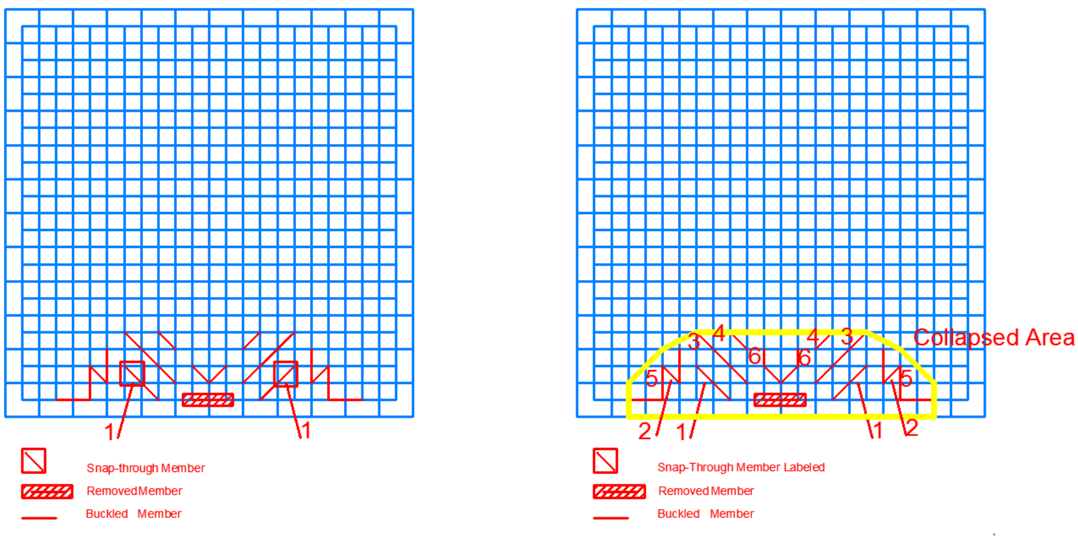


Figure 5.26 : Critical Results for Nonlinear Alternate Path, L1 Removed.

It can be seen that several of the diagonals in the area of removed lower chord member LI have buckled. In Figure 5.26, a symmetrical pair of those diagonals labelled 1 have exhibited chordal displacement snap-through, a member instability in which the force carried by a buckling member suddenly reduces and large chordal displacements occur. This phenomenonis due to the rapid rate of post-buckling unloading is combined with low stifness of the structure adjacent to the member. For the pair of members in Figure 5.26, the snap-through occurred, when the force in those members fell to 38% of their buckling load. The external loading could then be increased, when a second pair of members, labelled 2, snapped through. And then, six pairs of members have snapped through, in the order indicated, precipitated by the load redistributions caused by the snap-through of the second pair of members. These snap-throughs and the other buckling members allow a fold line to develop, such that the collapsed area becomes unstable.

It seems that compression member overdesign is a potential method of guarding against progressive collapse. In addition, overdesigning the lightly stressed diagonals in the middle half of the span along and adjacently parallel to the column line should also improve resistance.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Practical Application of This Study

For the practical application of this study, similar to the Turkish seismic design code the Turkish code for applied loads TS 498 [26] as well as design code for steel structures. TS648 [27] should be updated immediately by providing contour maps for 1h rainfall with a 100 year return period storm as well as by defining rain loads and additionally by giving the requirements against ponding and stability of the roof for the design necessities. Ponding failure must be taken as scenario during the design stages for the roof structure, and structural engineer has to trigger all the other diciplines to ensure safe and protected roof structure. Municipalities should consider regular check for preventing the roof from ponding failure.

As already mentioned in the previous sections progressive collapse makes the ponding failure as a catastrophic case. In addition, it should be underlined that not the ponding failure causes progressive collapse. For the developing countries likewise Turkey, progressive collapse potential for existing buildings as well as in designing new buildings to resist the collapses caused by extreme fire, internal explosion, external blast, impact or other kinds of terrorist attack, as well as earthquake and foundation movements, have to be evaluated and analyzed appropriately.

In my opinion, Turkish government should support the researchers to work about progressive collapse analysis, and progressive collapse should be even though included in the curriculum of civil engineering departments during the undergraduation education level. Since, during my study I have seen that there are so many structural engineers who have not come across with the phrase “progressive collapse”.

Moreover, progressive collapse analysis should be additional for the Turkish Seismic Code, in the first years it can be applied depending on the importance of the building.

Later on, it can be generalized for every type of buildings, since human life is the most important thing in every life cycle on the earth.

6.2 Future Recommendations of This Study

- Instead of Murtha-Smith's (1988) alternate path method methodology for both linear and non linear analyis any other methods can be used for the comparions, the new method can be developed or found by literature review,
- Nonlinear analysis program likewise ANSYS, ABAQUS can be used,
- A rate of convergence for stable systems to determine a safety factor can be calculated,
- For protection from progressive collapse, compression chord members and diagonals along and adjacent to the column line should be overdesigned or not can be analysed,
- Boundary conditions, member types, span- depth ratios can be examined,
- Vulnerability of members can be examined,
- Dynamic effects when a member is lost while the system is under over load can be examined,

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APPENDICES

APPENDIX A

1968 Ronan Point Collapse [28], [29], [30]

Ronan Point was a development of apartment buildings in London. It was built between 1966 and 1968. On the morning of May 16, 1968, a gas leak caused an explosion in an apartment of the 18th floor of one of the buildings. The explosion blew out an exterior wall panel. The loss of an exterior wall triggered the collapse of the upper floors followed by the collapse of the floors below due to the impact of the falling upper floors.

The Ronan Point buildings were 64 m tall, 22 story apartment buildings. With five apartments per floor, the footprint of the building with the collapse measured 24.4 m by 18.3 m. Of the 110 apartments in the entire building, 44 were one-bedroom apartments and 66 were two-bedroom apartments. The structural system, including the walls, floors, and staircases was precast concrete. Each floor was supported directly by the walls in the lower stories. The wall and floor system fitted together through slots and were bolted. The connections were filled with dry packed mortar to secure the connection. The system used in Ronan Point was selected because of ease of construction. The structure was assembled by lifting the precast concrete panels with a crane and then bolting them together. In essence, the structure was like a "house of cards" with no redundancy for load redistribution in the even of a local failure.

On the morning of May 16, 1968 a gas explosion blew out an outer panel of the 18th floor of one the buildings in Ronan Point. The loss of a bearing wall in the 18th floor caused the progressive collapsed of floors nineteen through twenty-two. Then, a second phase of progressive collapse occurred. The dynamic loading imparted by the falling debris triggered the progressive collapse of floors seventeen and below. The southeast corner of the building collapsed to the ground level. The collapse destroyed the living room portions of the apartments, leaving intact the bedrooms, except for floors seventeen through twenty-two.

Figure A.1 : Ronan Point After Collapse

The British government formed a team to investigate the causes of the collapse of the Ronan Point Tower. The investigating team concluded that the explosion was small and estimated that a pressure less than 10 psi was originated from the outburst. The primary evidence of a small explosion was that the hearing of the person who lit the match was not damaged. Tests were performed by the Building Research Station and Imperial College to estimate the structural capacity of the as-built Ronan Point tower. The results showed that the kitchen and living room walls would fail at a pressure of about approximately 11.7 kPa, while the exterior wall would fail at a pressure of approximately 20.7 kPa.The collapse of the Ronan Point building was attributed to its lack of structural integrity. There was no alternate load path for redistribution of forces at the onset of the loss of a bearing wall. Therefore, as the exterior wall of the 18th floor apartment was blown out, the exterior walls of the upper floors immediately.

Description of Collapse Analysis of Space Trusses (CAST)

Nonlinear Member Response [22]

The collapse analysis of space trusses (CAST) program requires that the member force-deformation response be defined. The response has been developed in terms of the chordal force-deflection relationship, in which the chord is defined as the line joining the original position of the joints of the member. A simplified member model can be used for a compression member which assumes linear elastic behavior up to a maximum force given by empirical buckling formulas (AISC 1978), constant force behavior until a plastic hinge is formed, and, finally, post-buckling softening behavior. For a tension member, linear elastic behavior is assumed up to the yield force, and then constant force behavior is assumed until rupture occurs.

Post-Buckling Behavior

When sufficient chordal force is applied to the member, full yielding will occur at one cross section. For a member loaded in tension, the member can elongate under no change in force until the onset of strain hardening. For a member loaded in compression, the full yield is assumed to occur at midlength, where a plastic hinge is formed. The full plastic moment of resistance will not be developed because some of the fibers are required to maintain equilibrium with the chordal force. However, the reduced momentof resistance will be approximately constant, and thus, if further chord shortening is to occur, the chordal force must reduce so that equilibrium can be maintained. The end chordal displacement u has components due to the rigid body motion and to axial and flexural effects. Equilibrium requires the following equation A.1.

$$P\,e_p = M'_p \tag{A.1}$$

Whereas P stands for chordal force, e_p is transverse displacement from the chord line at the plastic hinge and M'_p is net plastic moment of resistance, L is length of member, AE is axial rigidity. The displacement u is then given by;

$$u = L(1- \cos\theta) + \frac{PL}{AE}\cos^2\theta \tag{A.2}$$

Reversals and Unloading

Tension yielding members or post-buckling compression members may wish to undergo reversal, i.e. chordal displacement sense reversal, often loosely called "unloading." Members that are partially inelastic, but not fully yielded to "unloading," will revert to elastic behavior with the original modulus of elasticity.

Member-Structure Interaction and Chordal Snap-Through Instability

The member is assumed to maintain equilibrium and compatibility with the adjoining structure. Thus, the member-structure interaction can be described in terms of chordal force-deflection equilibrium and compatibilityrelationships. A typical relationship between the nonlinear response of a member and of the adjoining structure to which the member is attached is shown in Figure A.2. Curve OABCD is the response of the member. Curves OEA and OFDB are responses of the adjoining structure without the member, subject to a given external load and changing chordal force in place of the member. Equilibrium and compatibility exist between the member and its adjoining structure. It can be seen that under some circumstances for a given external load, two solutions might exist, e.g., at B and D. Thus, as the structure is loaded, point B will be reached.

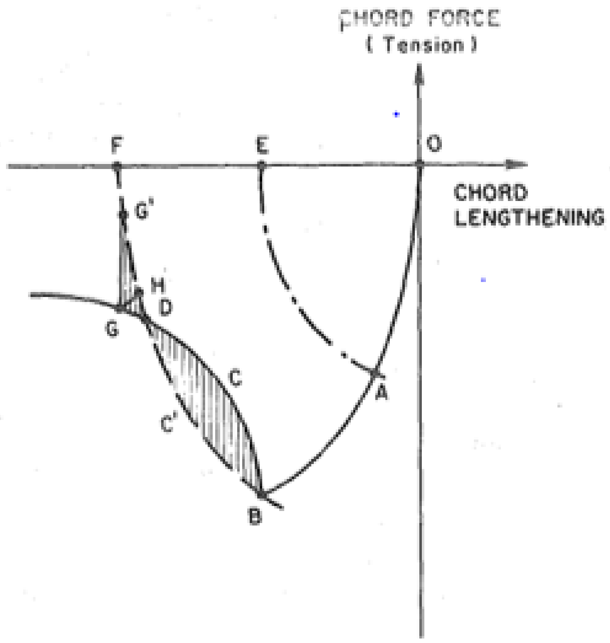

Figure A.2 : Member Structure Responses, Chordal Snap- Through.

Point B is an unstable point, since a slight increase in the chordal displacement will cause the force from the adjoining structure to be greater than the equilibrium state force in the member for that displacement. Equilibrium cannot be maintained, and thus the member will continue to chordally displace or chordally snap-through with the adjoining structure until equilibrium and compatibility are again possible at point D. Work is done by the structure and is being partially dissipated in the member. The excess energy, given by the area BCDC between the responses of the member and its adjoining structure, will cause point D to be overshot to GG', until the energy is dissipated. Equilibrium and compatibility are restored after oscillation at point H. When the load on the structure is subsequently incremented, the member will return from point H to G and then down the softening post-buckling curve. As a simplification, the writer has neglected the dynamic effects and used point D as the lower snap-down point instead of H. It is believed that the error incurred by this simplification would generally not be large.

Nonlinear Collapse Analysis Methods

As members move through their respective nonlinear response curves, the stiffness of the members and the system changes. This behavior can be accounted for directly by updating the stiffness matrix which, given the number of iterations required to make a full and accurate collapse analysis, makes this method expensive.

An alternative approach [31] is to create an equivalent "initial stress" or "residual force" system, which, together with the external load acting on the original structure, will produce a set of internal forces and displacements that correspond to the nonlinear response. The residual force system is caused by applying to the joints of the truss fictitious loads whose magnitude is determined in some rational manner. Since the fictitous and external load systems are applied to the original structure, the stiffness matrix does not require modification and reinversion,and thus the method offers considerable computational run-time savings. The determination of the fictitious load system generally requires an iterative procedure.

CURRICULUM VITAE

Name Surname : Aslı ARAS

Place and Date of Birth : SİVAS - 08.10.1987

E-Mail : aslisarkisla@gmail.com

EDUCATION

- **B.Sc.** : 2011, Middle East Technical University, Civil Engineering Faculty, Civil Engineering Department

PROFESSIONAL EXPERIENCE:

- 2011- Project Engineer (ENKA- ÇİMTAŞ)