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Content

CHAPTER I

Stability consideration in the safety aspects of urban underground spaces

Mehmet Kemal GOKAY[1]

1. Introduction

Underground voids and spaces (natural or man-made) have been considered in different applications in a variety of manners. In natural gas, oil, and groundwater related research, micro-voids in rock masses and their inter-connections have directed the natural reservoir capacities. Dimensions and stabilities of macro-scale underground spaces like; caves, galleries, machine rooms, shelters, underground cities, urban underground spaces, etc. are also important considerations in human societies. Thus, parameters influencing underground space stability and safety in general concerns should be main governing issues in the design of their excavations and utilisations. Urban underground spaces could especially be facilitated as parts of museums (underground

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showrooms, depots, etc.) and transportation network of cities & countries (tunnels, galleries, metro stations etc.) which are going to be used by more and more people, commuters, need to be designed accordingly to prevent any kind of accidents causing injuries or fatalities. These underground spaces should also be kept under close monitoring-maintenance-service controls for their stability and safety precautions. Urban underground space, (UUS), brings new understanding in urban transportations and urban living-working spaces in the modern world. Creative designs and new technological applications lead this concept into futuristic space age stages. The decision parameters which are required to be handled for UUS stabilities are evaluated here to understand the work context of engineering.

Underground urban spaces have been used in different countries for different purposes. Civilisation history has included numerous man-made underground excavations together with natural caves as well. Excavated urban underground spaces have usually developed for protection purposes against climatic effects and other outside influences. Natural caves have also been explored mainly for their open underground volumes, accessibilities, protection features supplied since the early times of history. When the existing historical urban settlements are analysed, similar possessions could be observed. For instance, locations of historical castles and main historical cities have their own particular reasons for their safety precautions. Caves and rock cliff spaces, (niches, which have cantilever roof & open one side faces), had long been underground living spaces for humans in prehistoric times. In modern times, caves which supply convenient heights and stable volumetric underground spaces have still been in use for different purposes. In addition, archaeological works pointed out that wherever the rock masses are suitable, man-made underground spaces opened in them for permanent or temporary (seasonal) shelters for their urban necessities. Natural caves and cave systems have different 3D underground spaces in diverse networks. They have open and collapsed (probably) underground spaces. Some of these collapsed spaces are wide enough to form deep rift passages when their roofs collapse in karstic environments. Some types of underground space conduits might cause sinkhole type surface failures. In order to analyse the stability conditions of caves' accessible openings, collapsed parts of the cave systems should be explored and they must be evaluated for their instability conditions. Actually, the reasons for having underground spaces (in micro or macro scale) in different rock masses, their dimensions, 3D locations, shapes, interconnections, and stability conditions are required to be explored for further engineering operations related to them. Moreover, increasing their dimensions, sizes, due to the various natural causes, requires extra engineering considerations for their post-failure conditions on surrounding rock masses. These cause could be; induced stress-strain distributions, shape & dimension factors in their instabilities, strength behaviours of massive & fractured rock masses, fracture initiation & propagation features of the rock masses, like creep long-time loading outcomes, bioactivities, chemical reactions, groundwater dissolution and weathering effects, etc.

Natural canyons, sinkholes, karstic areas including cave systems are then best examples of collapsed underground spaces through time due to natural processes. Stable caves' dimensions, shapes and their surrounding rock masses "quality" (in different rock mass classification systems) have valuable hints for man-made UUS designs. Thus, caves have cases of evaluations; i) for instability circumstances, ii) for the factors influencing long-lasting stable underground openings. Analysing deterioration and reinforcement effects of natural processes in the rock masses surrounding the underground spaces have supplied long-time stability concerns on caves and man-made underground excavations. Solubility of the limestone rock masses through groundwater circulations sometimes provides good chemical conditions for their precipitations. These slow chemical processes provided new surface covering layers on some parts of cave walls (like fine sized shotcrete). These procedures can possibly have suitable conditions to develop stalactites at cave roofs and stalagmite at cave floors. As the precipitation of calcium carbonate had continued in time, stalactites and stalagmites could be connected to each other in the cave to form roof support like mine pillars. Besides these rare cave features, most commonly gradual precipitations of calcium carbonate or any other mineral compound as filling matrixes for the apertures of the rock discontinuities surrounding the underground spaces provide upright rehabilitation means to them. These chemical reaction circumstances naturally occurring in time deliver clues for rock engineers to perform cementing applications for the fractured rock masses surrounding UUS. Cement slurry pumping (charging) into the fractured rock masses was already a common procedure (in certain cases, it is obvious ground engineering measures) for some types of rock foundations for dams, bridges, buildings, etc.

Time itself is one of the variables for mechanical & chemical interactional events in rock weathering processes. The other factors influencing the rock masses' weathering procedure; weathering characteristics of rock masses, dissolution rates of rock types, formation of sedimentary rocks especially precipitated from surface/underground water bodies, fracture initiations & propagation properties of rocks, rock toughness & creep behaviours of rock masses under long-time loading conditions, etc.

Surface structures and underground spaces in different parts of the world have been overlapped. These were predominately modelled in some cases of historical settlements. However, in modern times, these overlapping are mostly unrequired due to the complexities formed for the surfaces/underground structures. City plans and their opportunities have been embarrassingly recognised after experiencing population pressures and their requirements (infrastructures). In the modern world, city plans must include centres for social-cultural-sportive activities beside the business oriented offices, workshops, studios, factories (industrial parks) etc. It is already well known that their interconnections through surface and underground roads & rails are prime important for regional master plans. Population pressures and certain economic benefits are being factors to provide high-rise surface structures. Climatic factors and other economic enforcements in modern cities force engineers in recent times to design UUS for different purposes but mainly for underground metro lines and facilities. Extending the city limits with high-rise apartments (on the earth's crust) and UUS (in the earth crust) have required new understanding in engineering and governing procedures.

Societies have the capabilities to keep their welfare as long as the governing rules have supplied acceptable economic and personal development conditions for their members. Rules & acts defining surface land ownerships, their responsibilities, taxations, and protections (against natural impacts and outsiders) etc. procedure differences are parts of nations' identity differences. In modern times, the introduction of increasing numbers of UUS forces national governments to define land-ownership definitions in a 3 dimensional positioning system. Actually, classical land ownership definition supplied by Romans (the Maxim rules) defined an ownership of the particular surface land area (parcel) included the ownership rights for this parcel's below (underground) and above (sky) parts. However, countries which are accepting Roman land ownership concept as a basis for their legislative acts have diverse implications for the cases of underground mineral and energy sources. Development in aviation since introducing balloon force governors to put limitations on the Maxim rules as well. The dilemmas which have been observed among the land ownership right holders (including responsibilities) and UUS project developers & aviation sectors were reviewed by Gokay, (2023a) to present the complexity of current situations. Currently, countries have their diverse applications for land ownership certification for separate, apart, living volumes at high-rise buildings. Similar considerations might have tried to be defined for separate, apart, underground spaces as well. However, there are some extra stability and safety issues (obstructs) which must be considered for apartment buildings and UUS cases. Foundations of surface structures are influenced by neighbouring surface/underground structures. Because of these facts, foundation areas of surface structures are kept a certain distance apart among them.

Stability cases of any UUS influences the rock masses surrounding them. Instability conditions arose for a single UUS is not an individual problem. Failure of rock masses around a single

UUS or collapses of its roof strata is not a problem for that particular UUS. Underground instabilities around UUS are directly influencing stabilities of neighbouring underground and surface structures. Therefore, new engineering projects including excavation and construction activities (impacts of machine/blast vibrations, introductions of induced stress&stain conditions, long-term loading changes due to surface & underground geometry differentiation in positive/negative manners, etc.) should need to be engineered and regulated accordingly. Actually, subsidence researchers in the mining engineering field have their explanations also for any UUS caused collapses. Ground movements due to the shallow depth metro tunnels & their stations like the other UUS excavations can have their instrumental monitoring procedures. There are also numerical induced stress & strain analysis methods to visualise micro/macro displacements at progressive phases of USS excavations and operations.

In addition to these facts, project engineers of a new UUS (including metro tunnels, carparks, living&working apart volumes, depots, workshops, shopping centres, passageways, etc.) excavation should be ready to have decision complexities contradicting with already available surface/underground structures (engineered volumes). There may be additional dilemmas when the projected USS have stability concerns due to deep seated mineral and energy reserve operations. If there are legislative rules and acts concerning all possible technical & ownership related conditions of UUS excavations & operations, that means there are legislative procedures also to describe; shares, rights, duties, responsibilities, etc. Other cases always have their gaps between “UUS applications” and “available legislative Acts”. This situation seems to be “common” in human activities, however, if an accident occurs causing further stability problems for projected UUS and surrounding structures, engineers are then asked to explain the full situation in these legislatively unclassified cases. The concept in these kinds of problematic circumstances covers decisions on; what could be the reasonable engineering & legislative decisions if valuable surface/underground structures and natural reserves have impacted negatively (may also include accidents with fatality results).

Exploration of underground resources for the UUS other than the resources of groundwater, oil, gases, coals and minerals has defined new understanding in urbanisation. There are many decision parameters in UUS design and plans, (including; regional urban plans, architectural activities, artistic features, engineered utilities & machines, etc.) which should be considered altogether through professional group works. Most of the parameters related to UUS design procedure are coincided with other underground operations like in mining and tunnelling. Likewise, there are numerous uncertainties in rock mass & material characteristics together with their mechanical behaviours under short/long-time loading & unloading conditions in UUS design stages. Recently, classical design steps forwarding certain straight decisions have gradually been dropped to include decisions taken through uncertain characteristics of input data. This new understanding in engineering design has started to lead related legislative acts (engineering decisions include their uncertain characteristics due to uncertain properties of input data should be mentioned in legislative acts with logical reasoning-outputs-responsibilities). Eurocode 7 steps for instance introduce descriptions to rock & ground engineering designs by mentioning "uncertainty" cases for the similar engineered applications.

Admiral & Suri, (2015) explained, in their work, why engineers should think about UUS to couple with future requirements of urban societies. However, the first step to be considered in engineered structures is always the safety of the people who are using UUS for their daily life requirements. When the safety of UUS is under considerations, stabilities of urban underground spaces are primary important consideration before the other governing design parameters controlling the dangerous conditions like; fires, floods, explosions, gas discharges, radioactive light emissions, electric current shocks, machine malfunctions, ventilation problems, transportation problems, explosions, social unrest etc. When there is an underground space which is planned to be facilitated for public usage (metro line opening, carpark, shopping centre, museum, hotel, sport-social-cultural centre, shelter, etc.) the stability and safety rules should be defined accordingly together with ownerships definitions and responsibilities. Permission and responsibilities supplied through government offices, (by holding full responsibilities) construction activities of UUS and their surface structures should be well defined in legislative acts to suit their users' and commuters' safety issues.

2. Accident types occurred at UUS

Accidents which would have occurred during construction and operation periods of tunnels have their reasoning separately important. Underground events happened due to instability cases of underground spaces have provided certain preliminary parameters (if they have been monitored) before the accidents. These parameters are valuable ones to perform back-analyses to understand the phases of differentiation in rock masses before the events. The questions might be asked here; If these parameters are measurable? Why has the stability of the UUS have been impaired? The forwarding reasons of similar queries are based on mainly uncertainties in the earth crust (rock & soil masses) mechanical behaviours. In the cases of dynamic 3D loading&unloading circumstances of the rock masses around UUS and their mechanical property fluctuations & deteriorations in time might be the main factors in these instability cases. There may also be human factors like; improper UUS-designs and unsuccessful applications & implementations of the revised engineering designs, (due to unqualified employers/employees, improper machine park, unmatched installation materials, etc.). Engineers should supply careful approach to laboratory/field tests which are mentioning mechanical strength values *(or any other mechanical property values in "averaged numbers" in the hand-out reports without supplying "number of test samples and their standard-deviation values; how & where the samples were taken; who collected & transported the samples from fields to the laboratories; who was the responsible engineers deciding the tests; who performed the tests; which laboratory and what kind of test equipment were used in the testing procedures; Which engineering standards were followed during the tests, etc.).* There are also uncountable uncertainties influencing the strength of rock masses. Back-analysing of each instability events of UUS by different professions are then valuable assets for the post-failure evaluation of these accidents. Likewise, Sousa & Einstein (2021) supplied information about accidents that happened during tunnel constructions to evaluate their reasoning.

Similarly, Zhu etal., (2022) presented statistical analysis of 48 major tunnel construction accidents, (MTCA), that happened in China in 10 years from 2010 to 2020, (Fig. 1). Their analyses included case characteristics of accidents including; "classification, time, region, location, construction method, and risk source". They discussed safe construction issues of underground tunnels covering;

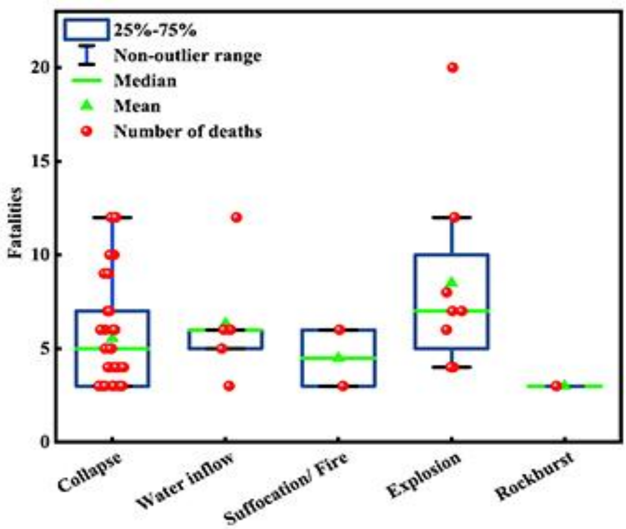


Figure 1. Types of major tunnel construction accidents (MTCA) and the fatality numbers. ("MTCA was defined as an accident that resulted in more than three deaths in China from 2010 to 2020" and "the red dots represent the different types of accidents and the corresponding distribution of deaths"), (Zhu, etal., 2022).

i) "How to accurately evaluate the resilient network of hazards", and ii) "How to build an efficient monitoring-mitigation-rescue system". These authors wrote that the most frequently occurring accident type reported in their research was related to collapsing (*31 accidents resulting in 172 deaths*). In underground operations, collapsing of roof and wall rocks of underground spaces are usually based on stability failures that originated through progressive events (developing at fast/slow rates). Zhu etal., (2022) pointed out that "the explosion related accidents" were the one that caused more fatalities. Their evaluation for the "*common factors leading to accidents can be classified into geological factors, human factors, and other factors, such as heavy rainfall or snowfall*". According to these authors' research, nearly 40% of MTCA were *"induced by human-dominated factors*". They added also that the remaining ratio part of MTCA (60%) was "*directly related to the insufficient prospecting, poor monitoring, and improper rescue*" activities.

Analysing and evaluations of safety concerns related to underground spaces including different types of tunnels have pointed to the vitality of the legislation acts and regulations with comprehensive contents to follow in UUS constructions and operations time periods. The numbers of commuters using tunnels in the world have regularly increased due to their particular advantages in certain business areas. These numbers have forced engineers & UUS managements to count the total number of commuters using their UUS facilities (tunnels, passageways, etc.) by different methodologies like image processing techniques. Actually, (with regular time intervals), monitoring these numbers and commuter distributions (densities) at different parts of the UUS for 24 hr time basis supply information to re-evaluate and plan rescue operation alternatives for possible accidents. Li, etal., (2024) wrote about the conditions and risk of rescue efforts of metro tunnel evacuation for the case of urban flooding disasters. Truly, tunnel design procedures should definitely include different UUS related accident case analyses to provide alternative emergency exit points and ready to use rescue apparatus. Bettelini (2020) outlined "*Common aspects* *and differences between the various types of “conventional” and “emerging” underground infrastructure*” and he discussed their consequences. Basicaly, when the underground spaces are categorised according to their usage types, the following list could be the one usually reached; i) Natural caves, ii) Rock porosity related reservoirs, iii) Mine galleries & openings, iv) Tunnels (for road, rail, and metro networks), station spaces, and machine rooms, v) Passageways, shopping facilities, and carparks, vi) Urban underground spaces, vii) Other emerging usage of underground spaces including deep storage reservoir & depots. Safety concepts related to these underground spaces have their individual requirements, but there is always a common approach in the UUS concept to keep the stability and access of the underground spaces as long as possible. This fact is also important for the rescue operation cases of possible disastrous events in UUS.

In general, UUS projects have mainly 3 phases, (an engineering point of view). These are; i) Exploration & design phase, ii) Excavation phase, and iii) Operation phase. The first phase predominantly influences the other successive phases, so all professionals employed for a new UUS project should be ready to use their full creative design capacities through collected and officially documented field data. Sub-decisions supplied by engineers and all the other professionals in these 3-phases must be documented with their reasoning. It is always better to keep the official second copies of these documents at notary safes as well. In case of any argument among the government offices, main contractors, subcontractors, employees and employers, etc., those documents including input data and supplied sub-decisions will be valuable traces of events at different worksites of UUS projects.

Bettelini (2020) stated that major safety steps have started to be applied and they have gradually been upgraded after large tunnel fires "*including the Mont Blanc tunnel (1999), the Tauern tunnel (1999) and the St. Gotthard tunnel (2001)*". He summarised main regulation progressive efforts to increase the safety of road tunnels while stating *"road tunnel safety is characterized by a thorough understanding of the infrastructure (tunnel structure and equipment) and improving operation but only an indirect and rather weak control on users and vehicles".* Similar statements could then be applicable for other UUS as well. Accident types provided in Fig.1 and other safety issues of UUS can then be related to; a) UUS stability concerns, b) UUS's facilities (equipment, machineries, instruments, supports, refurbishments etc.), c) Operational failures & accidents, d) Human factors (as workers, employees, commuters etc.), e) Implemented machine-vehicles-equipment-instrument types and their properties. etc.

Man-made underground spaces which can be abandoned underground mines or new excavation spaces have their excavation procedures. Abandoned mine spaces have to be evaluated through their earlier usages as parts of mining operations. Available mine records can be used in this type of applications to understand these spaces' mechanical behaviours. Abandoned mine spaces' long-term stability features are valuable assets during their evaluations for their secondary usage as UUS. After getting official permission for the operations of tunnels according to their projected targets, UUS's operational time period is started which should also be monitored for their stabilities and safety necessities. Operational periods of UUS (metro tunnels & stations, shelters, underground cities etc.) cover situations in which numerous commuters have their daily activities (travelling, living, working, etc.) in them.

Modern time living activities cover underground urban activities which need legal Acts to enforce, (engineered) precautions and rescue procedures. In order to visualise the vitality of these rescue readiness, the number of commuters in underground metro tunnels at certain time intervals should be considered deeply. If there is a tunnel stability problem in one of the metro lines in a city for example. If the length of the problematic metro tunnel influences 2 metro-train which has 10 cars (wagons) each, then around 2000-3000 commuters in each metro-train might be in dangerous situations underground. Authorities charged to operate the metro system in that city should be ready to handle all kinds of accident types together with alternative emergency exit paths and well-trained rescue teams. In each accident, there is always a certain level of a rescue operation, but readiness, organisations and effectiveness of these operations and number of rescued commuters will be the deciding factors here to speak about the quality of the supplied UUS services and its rescue understanding.

3. Parameter influencing stabilities of UUS

Bobylev (2009) pointed out some other considerations for urban planning in case of UUS utilisations. He wrote that "historic top-down development of Urban Underground Infrastructure (UUI) and shortcomings in its planning have resulted in a lack of available UUS for new developments". Surface land parcels defined through urban master plans have their categorised land ownership registrations. Surface land owner rights defined through countries legislations, and requirements of underground volumes (metro tunnels & metro stations etc.) for general public benefits have potentials to produce an ownership dilemma (Gokay, 2023a, 2023b). Bobylev (2009) categorized UUS types according to their functions (like in Paris-France, Stockholm-Sweden, and Tokyo-Japan, Fig. 2).

He pointed out similar complexity for land ownership cases between surface lands and underground spaces. He then emphasized urban planning that included the possibilities of UUI and UUS based on productive use. He wrote that; "A sectoral approach to UUS development and UUS availability on a first-come-first-served basis does not allow the full benefits that UUI can provide for urban sustainability". Since the requirements to the UUS is in increasing phase, planning excavations for surface and underground structures

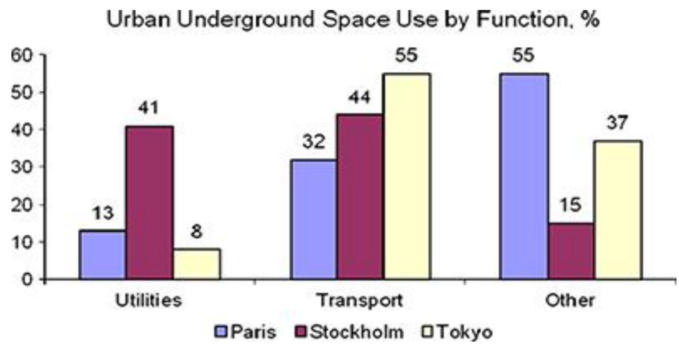


Figure 2. Types of urban underground space usages, *(Bobylev, 2009).*

have to be in controlled manners for practicing valuable natural, non-renewable, resources. Projects related with any surface and underground structures should then be evaluated through groups of professional (qualified) engineers & city-planners to provide efficient regional/local urban plans. Underground facilities excavated in earth crust have included disruptions of the rock masses at the project locations. It is impossible to eliminate impacts on natural resources (differentiation of: void dimensions, cracks, joints, groundwater conditions, stresses-stains distributions, etc.) due to UUI and UUS excavations. Their influences are mostly greater than the surface excavations. Introducing new UUS into certain urban locations supplements new complexities into legal land ownerships. Thus, volumes in earth crust as UUS should definitely have 3D coordinate positions to recognise coinciding (stay under) surface land parcels' positions. Additionally, explorations of already existing underground cavities under the city limits might provide new discoveries of caves, forgotten/undocumented shelters & infrastructures, archaeological underground passages/tunnels which have brought complex stability concerns for themselves and surface urban structures.

Similar studies performed for Singapore were discussed by Schrotter & Son, (2019) and they mentioned the requirement of a digital map of subsurface utilities. They reported the works (including georadar measurements) to obtain digital 3D maps for the measured areas of city land parcels. Actually extending these kind of 3D mapping projects to cover; more depths for instance (depth starting from official ground zero datum level) are going to be near future considerations in most of the urban areas where underground spaces (natural/man-made) are creating stability problems for surface structures (buildings, bridges, dams, etc.) and underground structures (metro tunnels, utilities, UUS, etc.).

Defining deep mineral, gas, oil, and groundwater reserves and their gaining methodologies have brought legal complexities for the surface/underground space ownership & responsibility concerns. When the ownership of underground caves, metro stations, metro tunnels, road & rail tunnels, underground shopping centres and passageways, underground workshop & depot organisations etc. are under consideration, due to additional gains in taxation issues, many shares are tried to be defined in legal & business activities. However, any shares or rights defined for underground spaces have limitations according to their underground positions. They might have other underground neighbouring spaces. They have certainly situated under particular surface land parcels (which have surface land parcel registration, land ownership rights) according to urban plans. Complexity in disputed ownership rights among them have potential to form deep dilemmas which should be dissolved by governing bodies of states collaborating through social acceptance. Admiraal & Cornaro (2016) wrote that legal land ownership rights, liabilities, and building codes were parameters to be evaluated carefully. Legal considerations of surface parcel and underground space ownerships are important for defining legal rights, taxation, and legal responsibilities adjoined. However, there are more important aspects related to them under stability considerations.

Rock mechanics covering mechanical behaviours and strength properties of rock masses; ground engineering (including mining engineering design features) plan and design activities push the companies and state offices to require well documented stability analyses of surface & underground structures before their utilisations and operations. Supplementary considerations related to the UUS & surface structures should be controlled before starting any engineering projects. It can be pointed out here that due to the significant influence of the UUS, new features of urban plans should be developed (introduced) to produce acceptable solutions for catastrophic ownership rights and structural stability problems due to the UUS inclusions. Actually, underground spaces have taken place in human life since the beginning of history. But, influences of surface structures and certain underground spaces' disadvantages, they have not been facilitated widely enough in modern urbanisation besides using them as depots and sheltering purposes. But, modern life in crowded cities forces societies to get involved with more UUS projects, (Fig. 3), wherever possible with their apparent advantages. Collecting data related to UUS's safe usage purposes include

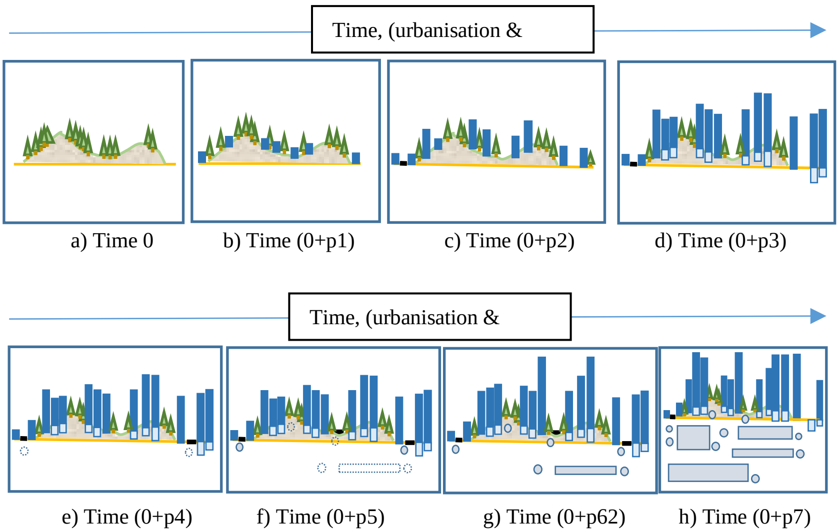


Figure 3. Urban city development and city limit extensions. Living & working spaces at the surface & underground structures.

parameters obtained at constructions and operation phases of them. Rock masses' property changes, dimensions of underground spaces, (micro&macro scales), cracks initiations and propagations monitoring at the main rock masses and applied supports, all applicant related UUS's operational parameters have to be monitored to predict any signs of negative influences on UUS's stability. Rastogi, (2008) was mentioned an instrumentation for instance to monitoring underground structures and metro-railway tunnels. Kushwaha, etal., (2019) worked on abandoned mine stabilities and mentioned their influences on surface urban structures. They wrote that *"railway lines, roads, buildings, villages, residential colonies, pipelines, etc."* could be over abandoned mine fields. These authors stressed on stabilities of these mines' abandoned openings and they wrote; "*the workings are generally old, abandoned and presently unapproachable. In some places, important delicate surface structures are installed above a portion of old abandoned underground workings*". Their works included numerical analyses concerning stabilities of underground openings. Subsidence caused instabilities at the foundations of surface structures could be explained (to some extent) according to the positions of underground spaces and surface foundations. The size of the surface and underground structures, weight of the planned or existing surface structures, disturbed zone around already existing underground mine space, depth of the mine, groundwater conditions, rock mass types between and their mechanical properties, time depended mechanical behaviours of the rock masses, etc. are the factors to be evaluated in subsidence studies. Actually, Longoni, etal., (2016) also mentioned the difficulties arose due to input data collection from abandoned mine sites (according to abandoned mines' stability conditions) for their further stability analyses. They wrote about environmental and stability conditions formed due to these mines by mentioning a census which has identified more than 2400 abandoned mines in Italy (Berry, etal., 2011).

In order to point out the impacts of old mining works on urban areas, the words of Longoni, etal., (2016); "geo-hazards related to the long-term stability of abandoned mines located in urbanized areas are a challenge for all countries with a mining history” illustrate the problems. Current mining companies have their engineered documentations describing mine operations’ environmental & stability influences on the surroundings. When the similar impacts for abandoned mines are under consideration, up to date data documentations are generally unavailable. In addition, official documents obtained through abandoned mine files are usually not comprehensive but they supply information about the extent of mine working sides. In order to collect current input data from abandoned mines to perform stability and environmental impact analysis, entering these mines may be thought of after obtaining official permissions. Safety concerns should be put on high alert levels in these types of research. Because, re-entering these mines’ openings to collect input-data for further engineering analyses is a dangerous act of researches, (dangers of rock failure & collapse around abandoned underground mine spaces and caves, for instance, have been continuous phenomena).Therefore, Longoni, etal., (2016) stated that “neither guidelines nor procedures are currently available for assessing how to approach this problem. In keeping with these environmental problems, long-term analysis of these sites should be planned”.

Stability & environmental effects caused by new underground metro tunnels might probably cause similar deformation cases. It should be borne in mind that there are always urban structures over the metro tunnels. Furthermore, there might be existing metro tunnels or UUS near the projected tunnels. In these kinds of complex decision environments, stress-strain interactions, stability features, existing subsidence trajectories, and influenced groundwater flow patterns should primarily be engineered for the new projected tunnels (by using the latest methodologies offered).

Analyses followed by researchers in mine engineering to define downward deformations during subsidence might lead the other professionals to evaluate similar conditions for new and old projects in tunnels and other types of UUS works. Lu & Han, (2023) studied the stability conditions of old metro tunnels. They wrote that “it is becoming more important to monitor and evaluate the performance and enhance the resilience of these structures”. According to them structural problems have been experienced during their service life. They also mentioned “excessive deformations, severe cracking and dislocation of joints” for structurally deformed existing tunnels. Supplying information only about the advantages of UUS, (metro tunnels, underground passageways and other underground spaces), in urban areas could be shortlisted if the long-term complexities about them are not mentioned. Progressive deterioration of rock masses around underground spaces lead further operational problems which gradually influences the efficiency and safety of the UUS. Thus, long-term performance of the UUS especially metro tunnels should be carefully monitored for the sake of operational safety concerns.

Dynamic features of the earth crust shape earth’s surface morphology by differentiating steadily rock masses types and their strength related conditions. Natural cavities and micro voids in the earth crust are expected to be formed and demolished in continuous manner along the geological eras. Urbanisation in human history is a very short time period when it is compared to earlier geological eras. Stable caves and micro voids explored in current times have provided information about their host rock conditions which might be formed in earlier eras. Thus, natural spaces which have been explored that are still open in current time might supply long-time stability preferences to the researchers. Similarly, if collapsed parts of cave systems are distinguished and the causes of defects in these collapsed parts of ancient cave networks are analysed, logical outputs can be obtained for engineers. What could be the reason for the collapse of natural underground cavities? How many years could these underground spaces remain open without any rock supports? The answers to these questions will contain valuable information that will be used to evaluate the instability conditions that existing man-made underground cavities may encounter in the long term. Beside, mine engineers also know that, collapses occurred at certain parts of a mine openings, (roof and wall rocks might have cracked and collapsed) trigger succession caving actions above that parts of the openings. Thus, failure of roof & wall rocks at mine openings is one of the major accident types in mines and unintentionally causes further complex stability situations in mines. Consequently, facilitating abandoned underground mine openings for their second usages as depots or as UUS have brought considerations about their stabilities. Some countries have their long periods of mining history which might cover accidents as well. Mine disasters, coal mine methane and/or coal dust explosions, and collapses at mine galleries have influenced those nations deeply. So they might be hesitating to use abandoned mine openings for their secondary UUS usages.

Similar underground space collapsing danger should also be under consideration for man-made UUS projects including metro tunnels. It was determined that collapsing type of accidents (Fig. 1) at underground tunnels was the most effective accident type in China in the 2010-2020 time period, (Zhu, etal., 2022). For metro tunnels; fast transportation activities in them require some engineering controls which should be realised continuously. These controls include; healthy of the rail-tracks, tunnels' ventilation conditions, stiff connections of all the tunnel applicants, vibration related aspects, groundwater conditions, fume & fire related dangers, explosion circumstances, flood & groundwater inflow cases, and additionally micro-scale displacements due to induced stress level surrounding the tunnels. Engineering projects related to new underground metro lines are currently introduced to urban areas in different ways, (with celebrations). Officials might mention the advantages of the metro transportations. They may also spend words about savings handled by metro tunnels, (minimising surface land compensation costs when it is compared the costs of mandatory surface land ownership right transferring for the cases of surface roads or surface rail-line projects) without mentioning instability concerns formed due to metro tunnels. Actually, officials and engineers should supply information about the advantages and disadvantages of the metro tunnels & UUS together. Because following facts; Surface subsidence expected (even in minor levels) during UUS excavation & operation periods; Sudden collapse occurred at different parts of UUS due to surrounding rock mass instabilities; Fires & explosions occurred due to equipment, their installations, and related human factors problems; Groundwater levels and related flow differentiations; etc. should all be considered to evaluate advantages & disadvantages of the projected metro tunnels and UUS. These evaluations need engineering group studies supplied by city planners, architects, archaeologists, engineers (civil, mining, rock, soil, foundation, geotechnics, geophysics, survey, computer, etc,), and officials from municipalities & central governments. Experiences gained in underground mining engineering have definitely positive impacts on these excavation &

monitoring subjects. Because underground mine openings with/without supports staying open in several decades, or collapsing in certain days/weeks had provided definitely valuable data & experiences, if they were collected/recorded orderly.

Long-term stability concerns and performance of UUS structures, road & rail tunnels, and metro tunnels are under consideration through several researches covering their; operations, controlling facts, maintenance & service activities, safety related procedures, stability analyses, etc. After construction of UUS including metro tunnels, what could be the monitoring steps and data collection actions related to the "health" of these underground facilities? Answering this question requires engineering group efforts. For instance; Schubert, (2015) defined why tunnel "health" monitoring is essential by pointing to the facts related to; Successful applications of observational approaches; Requirements of measurements (data collections); and Types of parameters monitored (i.e; displacement monitoring, methodology differences). Deformations realised due to induced 3D stress conditions around the UUS, for example, could be monitored by extensometers, optical sensors, strain gauge attachments, etc. A method supplied by Dinis da Gama, (2004) combined electrical & electronic engineering knowledge with mining & rock engineering analyses to monitor strain gauge resistivity differentiation. These gauges were attached to steel supports of underground excavations to monitor their strain differentiations. Most of the UUS, metro tunnels & stations, and underground public passages have their concrete supports and concrete linings. Controlling & monitoring micro structural events on these linings, (fracture detections, concrete material deteriorations, tunnel dimension deviations, micro-displacements, groundwater seepages, etc.) have provided input data to be used for underground spaces' stabilities together with rock mechanic test results of surrounding rock masses.

Stability monitoring of the UUS including metro tunnels & stations are crucially important for societies. Any inconvenient accidents related with tunnel supports or overstressed tunnel parts will produce partial collapses of UUS which include a large number of commuters to be rescued in a short period of time. Therefore, continuous monitoring the UUS are important as the case for their excavation time-periods. Lu, etal., (2012) are the researchers who have focused on submarine tunnel stability cases for instance to supply a long-term monitoring software to help engineers during their "long-term monitoring data acquisitions in real time, data analysis, and structure safety decision and pre-warning for submarine tunnels". Xie, etal., (2013) also pointed out the importance of the tunnels' continuous health monitoring systems. They wrote that Micro Electro-Mechanical Systems, (MEMS), which could be merged wherever required and then their "wireless sensor network" are then used to collect input data from tunnels (UUS). Another researcher group, Li, etal., (2014) forwarded a tunnel structure safety monitoring system based on BOTDA technology. According to these researchers, conventional tunnel monitoring systems including strain gauges, deformation measurements, etc. are not effective in continuous measurements of "the property differences" in structural integrity of the surrounding rock masses of USS. Based on fiber optic property changes under the influences of temperature & axial stain differentiations, BOTDA technologies featured this fiber optic property fluctuations for tunnel health monitoring system. The authors wrote that "optical fiber in the backward Brillouin scattering frequency will drift” due to “the optical fiber strain and the temperature variation”. Since there is a linear relationship “by measuring the frequency drift of Brillouin scattering back in the optical fiber”, it provides information about the change of “temperature and strain distributed information along the optical fiber”. This technology has further opportunities in monitoring stability conditions encountered in all types of underground openings. Di Murro, (2019) performed a thesis study for the long-term performance of concrete support linings of the tunnels at The European Organization for Nuclear Research, (CERN, Conseil Européen pour la Recherche Nucléaire) labs. In this study it was aimed to “develop a good understanding of the long-term tunnel lining deformation mechanism when its tunnel drainage condition changes many years after construction”. The thesis study included works to point out tunnel lining deformation mechanisms by means of observations (crack developments inner periphery of the tunnel). The author used “different monitoring technologies and interpreted the data to assess the mechanism of tunnel lining deformation”. Frenelus & Peng, (2023) were also focused on long-term monitoring of the structural health of deep rock tunnels.

In rock engineering concept, comparing field (in-situ) measurements with the outputs of numerical analyses is a common methodology in studies. Similarly, Yertutanol, etal., (2020) mentioned their displacement measurements data in twin road tunnels in Izmir, Turkey. Monitoring the tunnels through vertical deformations at the tunnel peripheries at selected locations have a common methodology to start understanding deformation characteristics of the rock masses and tunnel supports surrounding the tunnels. The data collected then used to evaluate further stability problems described through the rock engineering concept. These authors compared their field measurement data with the tunnel displacement data obtained through their digital displacement analyses conducted through finite element analysis. They reported that 70% of the calculated displacement values obtained from their numerical analyses were matched with the displacement data obtained from the field measurements.

Damages and deteriorations which have occurred on UUS supports provide visible or invisible signs for human eyes. Micro cracks traced on the surface of the concrete support rings of circular tunnels might be the visible parts of many other branches of invisible cracks propagating through the tunnel supports. Concrete supports, concrete columns & beams, steel arch type supports used for UUS are sensitive 3D stress distributions over them. There are fluctuations in these stresses in time and their continuous long-term impacts on UUS supports are vital to be monitored. If it can be presented in categorised manners; Long-lasting unchanged rate of overburden loads; Fluctuations of 3D vertical stresses due to developed stress conditions; Vibrations loads formed due to heavy-weighted machineries (metro trains, traffic loads, electrical supply related power transformers, etc.); Vibration due to earth seismic events, have their negative influences on UUS's supports and their surrounding rock masses. They might be the reasons for crack initiations or crack propagations in these supports and rocks. Therefore, monitoring cracks at UUS supports and surrounding rock masses have their significant signs to predict further fracture behaviours and their resultant displacements. It was tested that applied low levels of electrical potentials on a solid volume produce electrically conductive potential layers over the solid volume's surfaces. When the electrical conductivity distributions formed due to these conductive potentials are measured with supplied net-work frames (which have equal distance among its notes), differences in measured electrical potential levels are then coincided with the local disturbances on these solid volume materials like; fissures, cracks (visible/invisible) etc. (Gokay, 2016). Underground space safety parameters influencing their operations including their stability cases are tended to be monitored through organised, automatic sensing methodologies. Zhang, (2020), supplied a study which could be categorised in this technology trend, and their study presented monitoring field activities by using Building Information Modelling, BIM. Similarly, in order to monitor the visible crack formations and their propagations through digital high definition images at tunnel supports Li, etal., (2021) provided an automatic, “metro tunnel surface inspection system”, (MTSIS). The authors reported that, MTSIS included several components to capture high definition images from the selected surface areas of tunnel supports. The systems had computer systems together with software to realise the monitoring works with suited data collection & processing procedures. They wrote that, they “propose to use a convolutional neural network for metro tunnel surface defect detection”. Gong, etal., (2021) also proposed an image processing step for automatic visual inspection of cracks on concrete surfaces of tunnel supports. Actually image analysis usage in this field of engineering has an increasing trend specially for images captured in a certain specified period of time. Documentations which are filed (high definition images) along the tunnels or other UUS support surfaces (especially concrete supports) provide time-basis documentations which could be analysed for further tunnel stability problems. In this common basis of study, Gong, etal., (2021) worked “on-board image acquisition system using multiple line scan cameras to capture the full-section images of tunnel surface” for crack detection purposes.

Ye, etal., (2021) worked to determine defects at tunnels in China. They wrote that they collected defect related data from 90 highway tunnels. They analysed these data then to define their characteristic outcomes in the following groups; “structural deformation, structural damage, material deterioration, water leakage, frost damage, construction defects, and others (accessory facility defects)”. Among the defects; lining cracks and water leakage were determined the most common ones in these data sets. The authors also wrote that one or more defects could be observable in a single tunnel. According to their studies, 72 defect causes were related to human-factors in 79 tunnels where the data had been collected for inspections. Actually, on the basis of their studies, human-factors are the primary cause of defects in highway tunnels in China. The second cause of defects was non-human factors, (55 defects related in this category in 79 inspected highway tunnels). These studies on tunnel monitoring content reveal that, stability of tunnels and underground spaces have their own excavation and operational conditions which influences the defects observed along their lengths. At his point categorising the general cause of defects at UUS in 3 main classes is meaningful. They could be related to i) Physical & chemical properties and mechanical behaviours of rock masses, ii) Operational factors (influence of designs, excavation steps, support types, ventilation, water discharges, operational conditions in tunnel service life etc.), and iii) Primary & induced stresses conditions. The works mentioned above mainly provided reasoning for one of these 3 factors impacting immediate or long-term stability cases of UUS including tunnels.

For the tunnel stability case examples, including the effects of rock mass properties, Liu, etal., (2022) work can be counted. They worked on “long-term safety and in-service durability” of highway/railway tunnels in squeezing rock conditions by analysing the field data collected from the Lianchengshan Tunnel, China. Data related to this tunnel was collected by monitoring activities in real-time due to severe large deformation conditions in construction periods. The authors wrote that, “the structural stress of the tunnel is not stabilized during the operation period but fluctuates periodically with an annual cycle”. They concluded mechanical properties of squeezing rock mass influencing the stresses around the tunnels which any modelling approaches should include their influences.

Continuous data measurements are organised for long-term stability monitoring systems of underground spaces. In order to compare displacements differentiations along a metro tunnel or any other types of UUS input data collection and their transferring & storing at the specified centres should be handled accordingly. Displacement measurements and their further analysis might force engineers to collect more input-data about the rock masses at the back of the concrete support linings as well. Basically, types of rock masses, their mechanical properties and behaviours have to be worked out for the underground spaces’ stability concerns. Actually, if there is a UUS project, stability concerns are one of the design considerations to compare planned UUS options. Then, stability related input-data collections and their analysis are continuously performed at all man-shifts of the project including (excavation, construction, and operation phases). Monitoring UUS through these phases helps engineers to understand post-construction behaviours of the projected UUS. Strauss, etal., (2020) mentioned the types of underground tunnels and their stability, (health), concerns in terms of their operation phases. They forwarded that “monitoring of tunnel infrastructure is a very complex task, involving vital assets of the community”. They also stressed on the responsibilities of monitoring procedures by referring to the study performed by Bien, etal., (2019) for bridges.

When the design concept regarding to UUS under evaluation through proper planning stages, following parameters should be controlled for better engineering: a) New settlement lands with or without any surface structures, (which have involved minimum level of excavation depth for their foundations, they have to be analysed through their land surface morphology), b) Earth crust’s surface shapes at urban areas, c) Rock and soil types encountered at urban areas (their influences according to their depth), d) Climatic features through seasons, e) Biological activities, f) Water related features, (groundwater & surface water influences). City planners have their professions to collect preliminary data for their regional & city plans. In some cases, these plans are developed for the rehabilitations of already existing regional and urban settlements due the requirements developed in time for the efficient usages of surface and underground spaces. Due to the economic factors initiated by population and social & cultural features of countries, some cities have to extend their urban limits in/on the earth surface. Planning of new settlements, rehabilitation of earlier urban areas have actually been the signs of these continuously required; new living & working spaces, connection roads, rail systems, metro tunnels, etc.

Delmastro, etal., (2016) for instance forwarded their study and mentioned about surface land use constraints and options supplied by city planners above (high rise buildings) and below (usage of underground spaces) the ground surface. In their study they presented “major trends in Master Plans: integrating underground planning into the whole city Master Plan and producing dedicated Underground Master Plans”. Their work included; best practices in these applications, sectoral approaches (commerce, leisure, transportation, technical etc.), design guidelines and supporting technical approaches (GIS, georadar). At this point these authors exemplified Helsinki (Finland) underground space usages, (Fig. 4). Helsinki has an Underground Master Plan to manage the

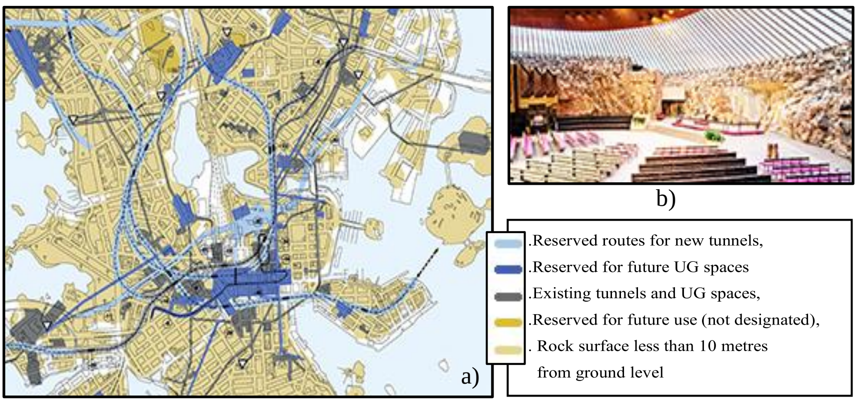


Figure 4. a) Part of Helsinki Underground Master Plan, “(Image: Helsinki City Planning Department), (Vähäaho, 2014a). b) Temppeliaukio Church (Helsinki, Finland) constructed through rock excavation and opened in 1969 (Photo:Juha-Pekka Järvenpää), (Vähäaho, 2014a).

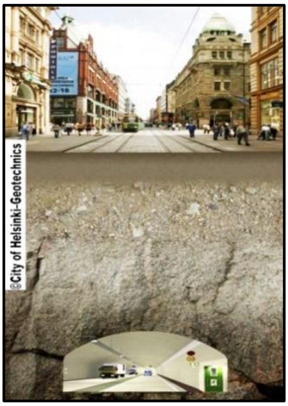
organisations of city utilities, spaces, resources for underground structures through the guidelines, (Vähäaho, 2014a & 2014b). When the subject is Helsinki underground facilities, it means complex underground space systems covering 10,000,000 m^3 underground volumes "*for parking, sports, oil and coal storage, the metro and so on*". Underground spaces in Helsinki were reported to include "more than 400 premises, 220km of technical tunnels, 24km of raw water tunnels and 60km of 'all-in-one' utility tunnels for district heating and cooling, electrical and telecommunications cables and water", (Vähäaho, 2014a).

4. Official registration data-documents related to UUS

Since underground space activities have gradually occupied more volumetric spaces below certain land surfaces, they are required to be recorded in an official manner to differentiate their rights and responsibilities. At this point, it is vital to clarify legal Acts & legislative procedures to identify (characterise) all types of operations for underground spaces. If there are mismatches in background Acts, these facts should be re-regulate through national assemblies by social agreements. Otherwise, problems that arise among surface/underground space owners will turn into endless dilemmas. Rock masses suitable for micro&macro underground spaces are national assets for countries. Thus, their usage must be regulated to maximise advantages of UUS and surface constructions for suitable rock reserves. In addition, due to their interconnected stability conditions professionals including engineers are forced to regulate their designs & plans. Combined efforts are then documented to provide long-term stability monitoring cases. If the concept is defining "*underground spaces through technical data & records (geological documents, mining activities, cave exploration data, etc.)",* there are available data management and graphic software. But, defining urban spaces (as underground/surface structures) in 3D models for their official records, there is no common practice yet. Because land ownership dilemmas (Gokay, 2023a) between surface land owners and underground spaces are not settled totally yet through national Acts (in most of the countries). Actually, there are uncertainties about who has the right to handle underground under the registered land ownership parcels for UUS cases. It is certain that excavating underground spaces (including all types of subterranean openings including metro tunnels & stations) have gradually caused subsidence effects on the ground surface. Therefore, surface land distractions due to new underground projects (shallow depth UUS developments, deep seated hydro-carbon extractions, groundwater extractions, or pumping out of natural gasses, thermal hot water extractions, etc.) have their induced stress-strain conditions on the existing surface/underground spaces. Therefore, there should be legislative Acts, rules, (cover the full scale of the concept without leaving any "grey zone" for further dilemmas) to protect the right of investors & contractors of UUS and shareholders of surface land owners (together with the underground space owners if there is any surrounding the new projected one).

Actually, national land ownership rules have to be re-arranged to handle the UUS. Countries generally have mining regulations, some have regulations for tourist caves, but most of the countries do not have regulations for the UUS. Theoretically, all the spaces (volumetric encapsulate) where humans or animals are living in/on earth surface should have registration number. Whoever has the right to use these spaces with ownership concepts (private or state owned spaces), they have their responsibilities as well. There can be no shareholder right which influences neighbouring spaces' stabilities. For example, surface land owners might have access passages to their private underground depots, caves, etc. (spaces) in certain parts of the countries (like in Nevsehir-Turkey, Nottingham-England, Italy, etc.), these surface and underground spaces are then re-defined and re-numbered for their legislative rights (if there is no covering regulation) according to proposed Acts near future. Countries which do not have rules to define underground spaces under the surface lands (or constructions) in official manner, they cannot supply pre-defined procedure to handle any problem arose due to these underground spaces. Lollino, etal., (2012) for instance stresses the stabilities of existing natural and man-made underground spaces to perform further engineering works in/on the rock masses around them. They pointed out that new projects related to urban developments (surface structures and infrastructures) sometimes are "*carried out without taking into account the possibility of encountering subsurface cavities, and the corresponding danger these might pose. In addition, loss of memory of man-made cavities under the historic part of many towns adds further problems".* In order to analyse stability interactions due to surface and underground structures, documentation which has been kept by official government branches, local organisations & societies, religious centres, and local people should be searched. At this point, any type of land ownership documentations, notes on these files, reports covering land morphology & geology, files including scientific papers & test results, academic documentations (thesis, reports, etc.), private companies' open access files-reports, oral-history related documentations, etc. should be screened carefully to reach a decision "if there is any evidences related to earlier underground space excavations below the specified land parcels". There is always possibility to have existing former underground spaces which had their historic records or they were totally forgotten due to their pre-historic excavation times, (or they might have totally elapsed due to their documents disappearances). These spaces could also be excavated for sheltering purposes in wartime. Thus, it is logical to have no (or less amount of) documentation about them. Some other underground spaces had been excavated to widen the underground network of official mine related openings, existing caves, depots, or UUS systems. If some of the mine stope tries or drift entrances had their 3D position changes after a few meters of excavations due to rock mass qualities, these tries and new positions of underground spaces should have mapped accordingly, any mismatches here bring unrecorded underground spaces to be considered in modern surface/underground structure constructions.

At this point, visualisation of underground through; earth resistivity, georadar-seismic-magnetic differentiation methods, etc. have supplied valuable manners to detect those suspected underground spaces. These remote mineral & energy resources exploration measurements have also been used to localise caves and man-made underground spaces, (including; abandoned mine spaces, archaeological underground spaces, etc.), In this field of application; El-Qady, etal., 2005; Pepe, etal., 2015; Kasprzak & Traczyk, 2014; Metwaly & Al Fouzan, 2013; Abidi, etal., 2017; Ungureanu, etal., 2017, etc. have their studies to be mentioned. Any method to have 3D documentations of surface land parcels and underground spaces together with their 3D coordinate interactions are very important to understand whole living & working volume concepts in/on earth crust. If these 3D documentations are in the form of digital 3D models, (which have subroutines to handle any kind of information for defined 3D locations in/on earth crust), the usages of such 3D modelling software programmes are very valuable. Information which could be supplied for certain earth crust locations through defined 3D locations, should be saved according to recorded date and include input values for; *rock mass types in depths, drilling logs, their mechanical properties, formation knowledge of rock and soil masses at those localities, geological activities through the geological eras, rock mass related faults, folds, thrusts, sinkholes, weakness zones, discontinuities, etc.* Software programmes which have been supplied data handling opportunities like the "Building Information Model", BIM, might be equipped with their 3D



History of *Underground Master Plan*

* Since the 1980s the City has maintained an underground space allocation plan.
* In the early 2000s there emerged a need to draw up an underground master plan for Helsinki.
* On 9 Dec. 2004, the City Planning Committee approved the planning principles.
* On 4-22 Apr. 2005, a participation and assessment plan was presented.
* In 2005 an open discussion event was arranged for anyone interested, and many detailed discussions were held with different interests.
* On 19 Jan. 2006, discussions were held with the relevant public authorities based on the participation and assessment plan.
* At the start of 2007 representatives were consulted from the water and energy utilities, police, defence forces and Helsinki City Rescue Department.
* In May 2007, draft plan made available, aim was to submit for City Council decision by end of 2007.
* 11 Dec. 2008 decision: revised draft to be resubmitted to Committee.
*On 17 Dec. 2009, proposed plan and the statements, objections, views and responses on it were submitted for approval by City Council.
* On 22 and 29 Nov. 2010, the City Board discussed the proposed plan.
* On 8 Dec. 2010, the City Council approved the Underground Master Plan of Helsinki (except for the reservation of Pitkäkoski water treatment plant, against which an appeal was made to the Administrative Court, but was rejected on 18 Nov. 2011).

a) b)

Figure 5. Managing, controlling, and engineering the urban underground spaces in Helsinki city, *(Vähäaho, 2012).*

modelling alternatives as well. These software will enhance the efforts of professionals (engineers, city planners, etc.) who have duties in regional urban plans including UUS and metro tunnels. Coordinating surface and underground structures with available rock mass reserve properties is an important governing duty to maximise the outputs of official regional and city plans. For similar focused achievements; European Cooperation in Science and Technology, COST, proposed "*European network*" system. COST have organised member states' efforts to form collaboration on "*Sub-Urban"* programmes to "*improve understanding and the use of the ground beneath*", (Schokker, etal., 2017). When the metro tunnel excavation & construction works are under consideration, there are works (related to urban plans, Fig. 5a) which should be performed by municipality officials before the project is put out to tender. Vähäaho, (2012) supplied for instance, a summary for Helsinki (Finland) underground master plan historic steps (Fig. 5b) which presented that Helsinki had UUS allocation plans since the 1980s.

5. Considerations of engineering risk related to UUS construction & operations

Underground works related to UUS have their differentiation due to their excavation and operation cases. When the risk related to work&workplace safety is considered for the handling stages of UUS including their long-term operational periods, all the professionals including engineers should be careful about their given decisions. Supplying information related with metro systems, Lin, etal. (2024) visualised content of the engineering work covers. The researches performed for the risk assessment of underground spaces are valuable information resources for engineering activities and decisions. At this point the study of Zou, etal., (2021) needs to be mentioned. After getting legal agreements on the land ownerships of surface parcels and underground assets, stability considerations are then analysed by taking all induced stress-strain interactions to coordinate overall subsidence effects in allowable limits. Induced stress differentiation caused mainly due to overburden loads could be the results of the following engineering actions; a) Subtracting earth surface load by surface excavation, (excavation performed for surface structures' foundations; open-pit mining operations; hillside excavations for highways, railways, and constructions etc.); b) Subtracting loads by excavating underground spaces in earth crust; c) Adding extra loads by constructing surface structures, (buildings, bridges, dams, high rise apartments, industrial plants etc.), d) Adding extra loads due to large water bodies collected at the dams' lakes; e) Adding extra loads (dynamic) due to short periods of earthquake waves; f) Stresses induced in rock masses due to the moon gravity (sequential loading & unloading). Engineers dealing with earth crust mechanical behaviours, (rock engineers, ground engineers, mining engineers, geomechanical engineers, civil engineers, geophysical engineers, etc.) have their experiences that 3D stress differentiation at the foundations of surface structures have developed gradually during their construction and then operation periods.

Similar influences have been realised around underground spaces which had been opened due to natural reasons, or excavated in engineering works *(mining, sheltering, passages, tunnels, living&working spaces, small&large scale repositories, etc. purposes)*. Ercelebi, etal., (2005) for instance supplied deformations (settlement, subsidence) analysis results obtained through 3D numerical finite element analyses for the selected part of a metro tunnel (Istanbul, Turkey). Similarly, Ercelebi, etal., (2011), Mahmutoglu, (2011), Chakeri, etal., (2014), Sojoudi, etal., (2021), provided their studies concerning the settlement originated due to twin metro tunnels at Istanbul and Ankara (Turkey). Settlement analysed for masonry buildings (residential complexes built in 1929) due to tunnelling analysed also through field observations and numerical methods exemplified by Andreu, (2015). Settlements and stability problems occurred due to tunnels and other underground spaces have usually been evaluated through in-situ measurements and numerical analyses methods by researchers; (Canakcı & Gullu, 2009; Wu, etal., 2015; Dai, etal., 2016; Liu & Ding, 2020; Aygar & Gokceoglu, 2021; Lu & Han, 2023; Yun, etal., 2024, etc.) to understand the risks of instabilities around the tunnels.

Living spaces in different types of houses, (in different urban settlements), and the land ownerships are historical issues through the states' tax policies. In some states, modern life is supported by required legislative Acts and governing bodies of urban settlements have their responsibilities also to supply public-sector duties like; supplying drinking water, collecting solid household wastes, planning-constructing-operating wastewater systems, etc. In some communities, heating systems are also included into the other public services. Realisations of these public service duties have their positive impacts on public health and welfare. What about the works which should be performed to have stable surface structures in urban settlements? Who is going to take responsibilities to direct communities in their selection of urban settlement locations. In general, connection roads availability, safety issues, population pressure, and positive opportunities of land ownerships have led the constructions of houses in cities through time. In later years, urban plans (regional plans, city plans, industrial area planning, etc.) were facilitated to organise urbanisation. Otherwise, for the cases of urban life without any plans, all the activities of human requirements are mixed up in settlements in chaotic manners. City planners have been educated to eliminate negative impacts of modern urban life by introducing planned lifestyle in well-organised, planned, cities. Next question for the planned urban settlements is; *"If the earth crust* *under the particular urban area is stable enough in acceptable limits?"*. In history, instabilities at the foundations of settlements had experienced through fatal accidents. In modern times, who is going to direct settlers if the land parcels have; slope stability problems, earthmoving dangers, and subsidence areas due to natural or man-made underground spaces.

For the public safety, local ground movements (wide area, micro level deformations) which will influence local people through their instable houses & apartments cannot be monitored through landowners' initiations. Actually, ground movement in micro-scale cannot totally be evaluated through analysing deformations at a single surface land parcel. Influences of subsidences and slope failures (ground movements in the whole area of the interest) should be monitored for the potential surface areas of the cities (where these locations could possibly cover many numbers of surface land parcels including houses, apartments, caves, and UUS). Therefore, susceptibility of ground movements should be performed by municipality officers (in some countries this work might be realised by government officers), with adjoining responsibilities. This kind of duties can be handled only by governing bodies which have their social responsibilities to their societies.

6. Conclusions

Underground spaces have different types and they have been used in human history since the beginning of human activities. Actually, natural underground spaces, caves, were earlier shelters for human societies. Underground passages and underground cities which are excavated for sheltering purposes from the climatic influences and other dangerous circumstances. In the earlier centuries, road and rail tunnels have been excavated for their positive advantages in transportation conditions wherever required. Metro tunnels, underground depots, shopping centres, underground warehouses, and carparks are some of the current usages of UUS. In crowded cities, members of societies have their experiences of underground spaces by using one of these (available) opportunities in their daily life. When the safety concerns are under consideration for these spaces, accident precautions (due to their usages) taken for UUS operations should be enough (as the regulations are ordered) like it is the case for surface structure usages.

It is also very important to consider the stability of the underground spaces. Like high-rise building, underground spaces have to be monitored for their early signs of induced 3D stress-strain levels before they are causing micro-cracks at their surrounding rock masses. Researches performed for UUS stability cases include following subareas of studies like; regional&city plan considerations; interactions of UUS with already available surface structures; monitoring the induced stress-strain conditions; monitoring groundwater conditions; measurements of natural gasses recharges from the rock masses to understand 2-phase (water&gases) flowing through rock permeabilities; estimation of flood dangers at the regions around the UUS; etc. It is important to point out that mentioning only the advantages of UUS, or the dangers of accidents due to UUS are not an objective working environment for engineers. UUS design, construction and operation phases should be analysed in engineering manners for their induced conditions in stress-strain and micro-subsidence circumstances. Thus, engineers have their responsibilities and duties to estimate optimum conditions for the projects including UUS and surface structures.

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CHAPTER II

Strucural Damage Detection in High-Rise Buildings Using Wavelet and Hilbert Huang Transforms

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1. Introduction

Highrise buildings must be able to meet the performance levels required by the earthquake and wind forces and supervise the manufacturing stages after they are manufactured. For this reason, there is a need for monitoring structures of high importance, heavy use of people and certain heights at the manufacturing stage and afterward, and regularly monitoring their structural performances. The basic principle used to detect damage in such monitored systems

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is changing parameters such as the frequency, mode, etc. in the structure by the damage that occurs in the structural elements.

As is known, Fourier transforms give a resolution in the frequency domain and are based on a specific mathematical background. With this transformation, modal frequencies can be determined. The tools used in the time and frequency environment are wavelet and Hilbert-Huang transformations. Wavelet is limited duration; its mean is called waveforms whose start and end values are zero. Since the wavelets are irregular, limited in duration, and asymmetric, they well describe the anomaly, vibration, and events in the signal. The Hilbert-Huang transformation (HHT) is used as an adaptive method that adapts to the analyzed data without using any sub-signals and produces different subfunctions in each dataset. Since it can not be expressed by any mathematical equation, the theoretical background has not yet been completed. HHT may become the most widely used intelligent signal processing system for in-situ monitoring of structures over the next few years.

Huang proposed a new method for analyzing non-stationary and non-linear data. This method consists of combining empirical mode decomposition with Hilbert spectral analysis (Huang et al., 1999). Zhang reviewed the Hilbert-Huang transformation's rationale for analyzing dynamic and earthquake records in seismology and engineering studies. This work has shown that HHD-based Hilbert spectra can accurately and temporally demonstrate the temporal-frequency energy distribution for motion recordings. In addition, in this study, peak ground speed (PGV) is a better measure of earthquake performance than structural ground motion (PGA). This is because the speed is more directly related to the low-frequency motion and energy that is proportional to the square of speed (Zhang et al., 2003).

Hilbert-Huang transformation (HHT) has shown that it can be used to define structural parameters in a comparison building. It is also the result that HHT consistently determines the position and level of structural damage (Lin et al., 2005). Salvino and Pines demonstrated the ability to extract phase information of empirical mode decomposition from discontinuous signals and used the results to detect damage of the structure (Salvino and Pines, 2006). Kareem studied the Hilbert transform by nonlinear signal characterization with continuous wavelet transform and empirical mode decomposition (Kareem et al., 2007a). In another study, Kareem evaluated the performance of instantaneous frequency estimates under noise by Hilbert Huang transformation with continuous wavelet transform. As a result, it has been pointed out that as the noise level increases, HHT's signaling and reconstruction processes from empirical bases are problematic, and that wavelet transform provides a more reliable alternative for such analyses (Kareem et al., 2007b).

Bao used automatic correlation functions as inputs to EMD for time-varying system identification and proposed an advanced HHT algorithm. This improved method is very sensitive to small vibration parameter changes applied to the modal identification of a composite beam scaled by various damage scenarios (Bao et al., 2009). Li investigated the relative displacement measurements of newly developed relative displacement sensors to detect the damage of the cutting connectors on composite bridges. Continuous wavelet and Hilbert-Huang transforms are used to analyze the measured dynamic response and to detect the damage of the cutting connectors in the compound bridge model under moving loads. Numerical and experimental studies have shown that relative displacement and acceleration measurements can determine the time and location of damage at the cutting connectors under loads on the bridge motion. The results show that relative displacement is a better response quantification for structural bridges (Li et al., 2015).

Ghazi proposed a new damage detection algorithm using the damage index and HHT. An important aspect of this algorithm is to use an energy-based method to combine several damage indices to reveal the linear and nonlinear effects of damage and reduce the probability of detecting damage. The raw signals of the structural response are the only inputs to the algorithm, and no information is needed about the geometry, configuration, and construction material. Experiments on the laboratory model structure have shown that nonlinear qualities due to damage in the structural response are effectively captured by the energy-based method. The Hilbert-Huang transformation is more efficient in extracting nonlinearities than the Fourier analysis but is seen as a computationally intensive process (Ghazi et al., 2015).

Beyen conducted damage diagnosis work in three different ways, namely frequency definition field, time definition field, and time-frequency definition field, obtained from a six-story reinforced concrete structure damaged in the Kocaeli earthquake of 17 August 1999. In this study, wavelet power spectrum features, statistical consistency, and cross-correlation applications, which constitute the movement point, have been shown to enhance the results obtained in structural health studies (Beyen, 2015). Guo and Kareem show that time-varying system properties such as temperature, aging, or extreme loads effect can be identified in output-only nonstationary system identification (SI) framework based on instantaneous or marginal spectra derived from the time-frequency representations (Guo and Kareem, 2016). Beyen evaluated analytical and experimental results in a one-to-one relation with transfer functions in the frequency domain and compared the observed error distributions with the interim or the data if possible, to determine the bearing wall, wall loads, simplified areal spread loads, and plastic hinge couplings used for damaged element rigidity it can be corrected (Beyen, 2017). Kareem offered a wavelet-based representation for a numerical example that demonstrates the estimation of nonstationary response using a generalized chain for understanding the wind effects on structures (Kareem et al., 2019). In this study, it is emphasized that the algorithms in monitoring, storage, and data processing in the light of recent technological developments are increasingly applicable to computers at high processing speed, but also applied in experimental modal analysis for civil engineering structures applied as standard in aircraft and spacecraft.

2. Theoretical Background of Time and Frequency Transformations

Frequency domain analysis can be done by Fourier transformation. Any function with this transformation is converted to the sine of the sum of the basic functions. Constructed signals *x(t)* are transformed from the time domain to the frequency domain by considering that they are influenced within an infinite period, from which the structural frequency information can be accessed.

$$X(\omega) = \mathrm{F}[\mathrm{x(t)}] = \int_{-\infty}^{+\infty} x(t) e^{-i\omega t} dt \tag{1}$$

In order to detect damage anomalies, wavelet transforms are widely used in time-frequency resolution. Wavelets have a limited duration; the mean of waveforms is zero and they start and end from zero. Since the wavelets are irregular, limited in duration, and asymmetric, they well describe the anomaly, vibration, and events in the signal. Damage detection techniques based on wavelet analysis are based on two different approaches. The first is discrete wavelet transforms. They detect sudden changes in the approach and detail levels in the signal. Rapid splashes in detail can be related to structural damage. As a second approximation, continuous wavelet transforms detect changes in the natural frequency of the structure by creating the time-frequency map of the response signal. In general, continuous wavelet transforms are better for time-frequency analysis, while discrete wavelets are better suited for decomposing, compressing, and feature extraction. In this study, continuous wavelet transforms are used for time-frequency analysis.

$$CWT(a,\tau) = \frac{1}{\sqrt{a}} \int_{-\infty}^{\infty} x(t) \psi(\frac{t-\tau}{a}) dt \qquad (2)$$

Wavelet transformations have several differences and advantages over short-time Fourier transforms (STFT). As is known, short-time Fourier transforms give time-frequency resolution, but there is no constant shape in the resolutions obtained from wavelet transforms. Wavelets have better frequency resolution at low frequencies and better time resolution at higher frequencies. This variable resolution gives an advantage when combined with low and high frequencies at certain frequencies in the same time-frequency domain.

Another method used in the time-frequency domain is the Hilbert-Huang transformation. After Huang's proposed decomposition, non-stationary signals are separated into empirical mode decompositions (EMD) and intrinsic mode functions (IMF) which are different from the initial signal. The Hilbert Huang Transform (HHT) is specifically designed to analyze non-linear and non-stationary data. According to Fourier and wavelet transforms, HHT is preferred to identify the instantaneous frequencies and amplitudes more locally. The HHT technique used for the analysis of data consists of two components, a decomposition algorithm called empirical mode decomposition (EMD) and a spectral analysis tool called Hilbert spectral analysis. The HHT can locally identify the components of a signal that are either non-stationary or non-linear. First, the EMD signal is transformed into intrinsic mode functions (IMF) and advances the scale characteristics contained in the signal. In the second part, the Hilbert transformation is applied to the IMF and a time-frequency representation (Hilbert spectrum) is obtained for each IMF.

Having obtained the IMF's components, it is possible to apply Hilbert Transform (HT) to each component, to get instantaneous frequency. The HT of a real signal x(t) is defined as

$$H[x(t)] = x^{*}\frac{1}{\pi t} = y \tag{3}$$

or using the convolution definition

$$y(t) = \frac{1}{\pi} P \int_{-\infty}^{\infty} \frac{x(\tau)}{t-\tau} dt \tag{4}$$

where P indicates the Cauchy principal value. From $y(t)$ it is possible to define the analytical signal $z(t)=x(t)+iy(t)$ or, in polar form, $z(t) = a(t)e^{i\theta(t)}$ in which $a(t) = \sqrt{x^2(t) + y^2}$, $\theta(t) = arctan\ (\frac{y(t)}{x(t)})$.

Where $a(t)$ is the instantaneous amplitude and $\theta(t)$ is the instantaneous phase function. Instantaneous frequency $\omega(t)$ is defined using the instantaneous derivation of phase

$$\omega = \frac{d\theta(t)}{dt} \tag{5}$$

With both amplitude and frequency being a function of time, we can express the amplitude (or energy, the square of amplitude) in terms of a function of time and frequency, $H(\omega,t)$. The marginal spectrum can then be defined as

$$h(\omega) = \int_0^T H(\omega, t)dt \tag{6}$$

where [0, T] is the temporal domain within which the data is defined. The marginal spectrum represents the accumulated amplitude (energy) over the entire data span in a probabilistic sense and offers a measure of the total amplitude (or energy) contribution from each frequency value, serving as an alternative spectrum expression of the data to the traditional Fourier spectrum. The signal can be written in the form

$$x(t) = Re\left[\sum_{j=1}^{n} a_j(t)e^{i\int \omega_j(t)dt}\right] \tag{7}$$

Table 2.1: Comparison of Transforms

Transform	Fourier	Wavelet	Hilbert-Huang
Basis	a priori	a priori	adaptive
Frequency	convolution: global, uncertainty	convolution: regional, uncertainty	differentiation: local, certainty
Presentation	energy-frequency	energy-time-frequency	energy-time-frequency
Nonlinear	no	no	yes
Non-stationary	no	yes	yes
Feature Extraction	no	discrete: no, continuous: yes	yes
Theoretical Base	theory complete	theory complete	empirical

http://www.scholarpedia.org/article/Hilbert-Huang_transform

3. Findings and Discussion

Twenty-three-story analytical reinforced concrete framework modeled according to Eurocode 8. Columns are selected as 80x80 cm^2 and beams selected 25x50 cm^2. The X and Y directional axes are 8,5m-3m-8,5m in length and the floors are 3,5m high. No moving load is exerted on the structure other than the natural loads. It was analyzed under earthquake and dead loads. The records of the Central Meteorology Station Directorate of Kocaeli earthquake were taken on 17 August 1999 as an earthquake force. Modal parameters were determined by certain floor stiffness reduction methods for comparing damaged and undamaged structural changes.

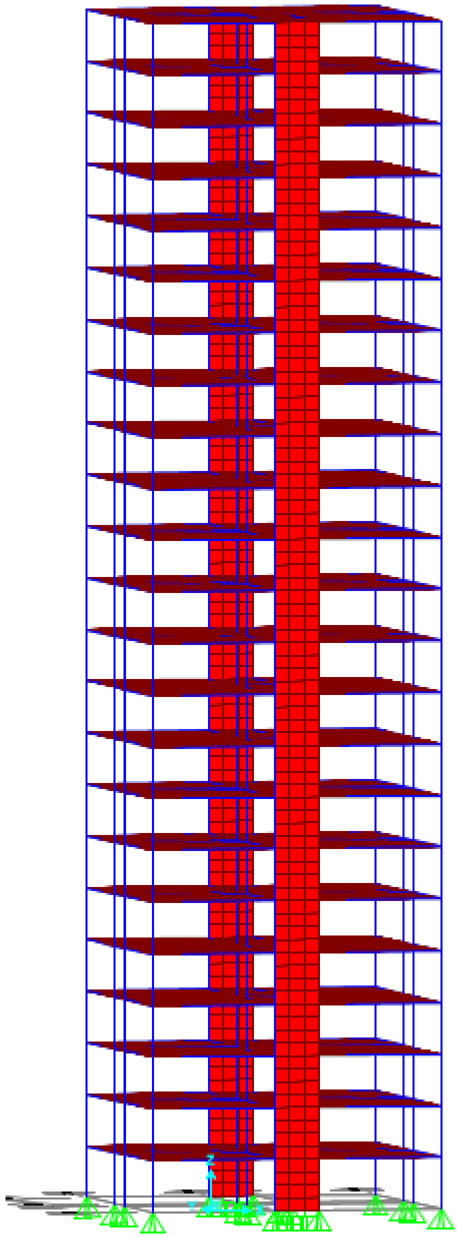

Figure 1. 3D Model of Structure

Earthquake analysis made by nonlinear time history case. Modal parameters of the structure are determined by the finite element method.

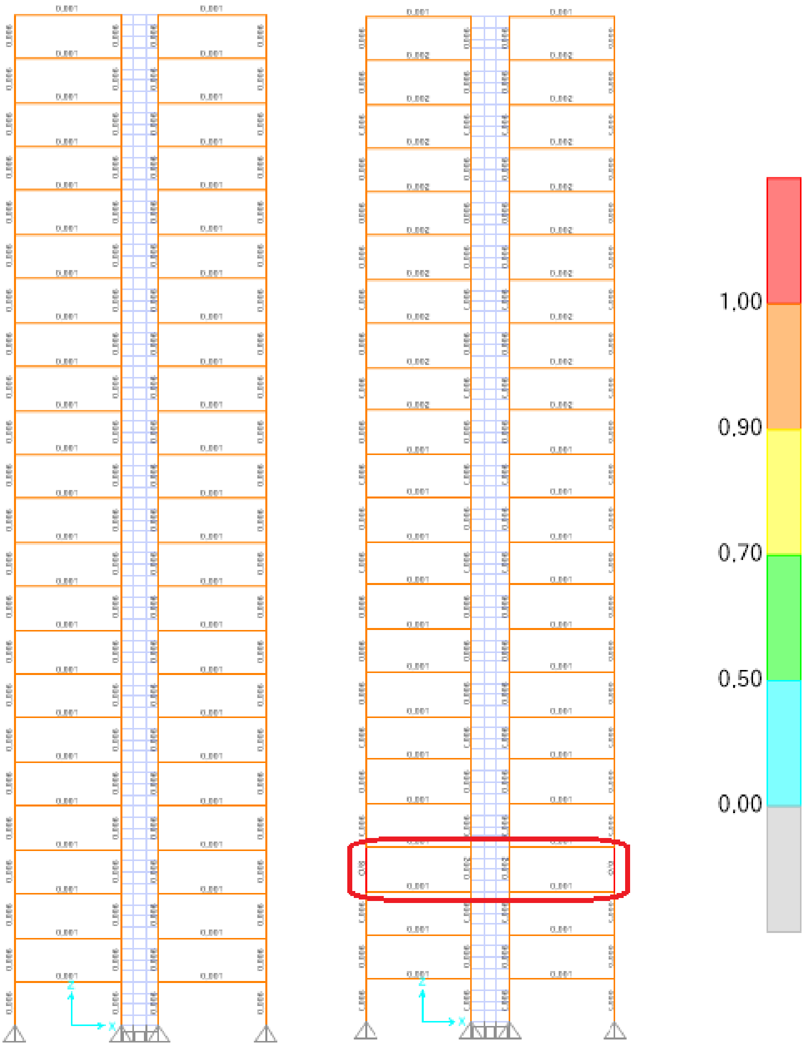

Figure 2. Performance Level of Undamaged (left) and Damaged (right) Structures

Table 2.2: Undamaged Building Period Determined by Finite Element Method

TABLE: Modal Participating Mass Ratios (Undamaged Structure)						
OutputCase	StepType	StepNum	Period	Frequency	UX	UY
Text	Text	Unitless	Sec	Hertz	Unitless	Unitless
MODAL	Mode	1	4,423	0,226	0,000	0,812
MODAL	Mode	2	3,515	0,284	0,699	0,000
MODAL	Mode	3	2,847	0,351	0,000	0,000
MODAL	Mode	4	1,407	0,711	0,000	0,110
MODAL	Mode	5	0,930	1,075	0,142	0,000

Table 2.3: Damaged Building Period Determined by Finite Element Method

TABLE: Modal Participating Mass Ratios (Damaged Structure)						
OutputCase	StepType	StepNum	Period	Frequency	UX	UY
Text	Text	Unitless	Sec	Hertz	Unitless	Unitless
MODAL	Mode	1	4,608	0,217	0,000	0,821
MODAL	Mode	2	3,639	0,275	0,704	0,000
MODAL	Mode	3	2,923	0,342	0,000	0,000
MODAL	Mode	4	1,446	0,692	0,000	0,101
MODAL	Mode	5	0,935	1,070	0,136	0,000

The dominant mode frequency for the X direction is 0.284 s in the undamaged model and 0.275 s in the damaged model. The parameters of undamaged and damaged states of these points were investigated by Fourier, wavelet and Hilbert Huang transformations and the differences between the two cases were tried to be determined. In addition, the transfer functions of the signals obtained from the floors were used to determine which layer of damage occurred. Among the obtained signals, acceleration, velocity, and displacement records are discussed which are more suitable data for damage detection.

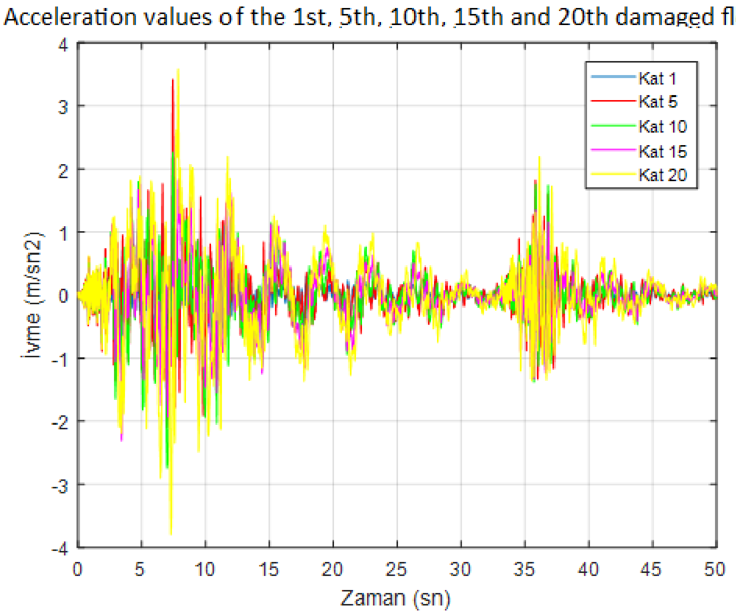

Figure 3. Acceleration-Time Graphics of Damaged Storeys

The acceleration time graphs of the damaged elements are shown in Figure 3. In addition, the Fourier transform is performed on the acceleration values of the 1st and 20th floors, and the modal frequencies in the x direction, 0.275Hz and 1,070Hz are found as seen in the first two graphs in Figure 4. It has been discussed that the first two modal freqs will come to a certain state by applying low pass, high pass, and band pass filters and the result is that the band pass filter is suitable as seen in Figure 5.

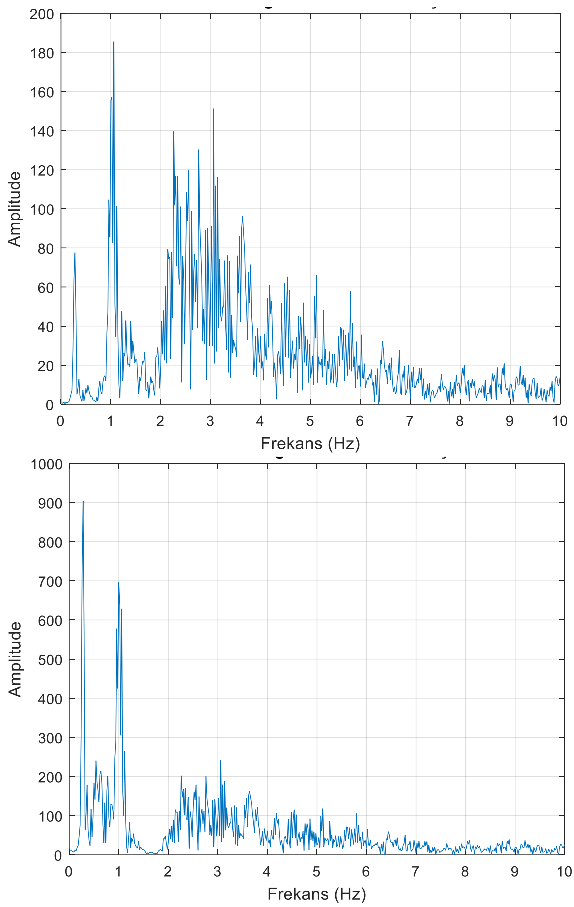

Figure 4. Fourier Transformations on Acceleration Values of 1st and 20th Floors

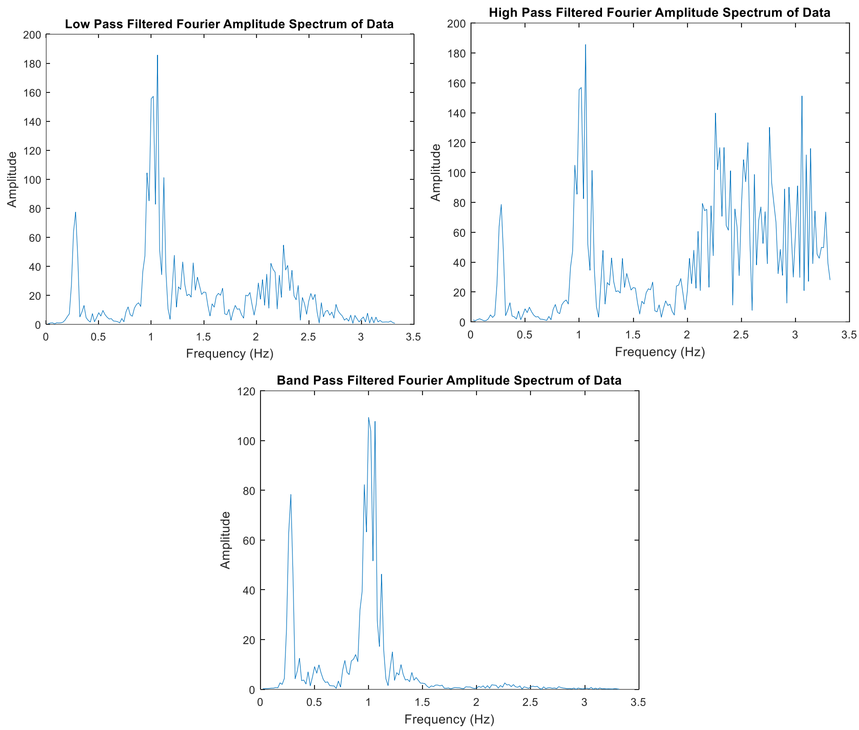

Figure 5. Low Pass (left), High Pass (middle) and Band Pass (right) Fourier Transformations of Filtered Data

When the spectral ratios between the accelerations between floors are taken into consideration, there is an opening between the graphs at the damaged floor and the floor below and it can be determined which layer of the damage is formed by this method. In addition, speed and displacement records have been used to accomplish similar tasks and as a result, displacement graphs are more successful in the detection of damaged floors.

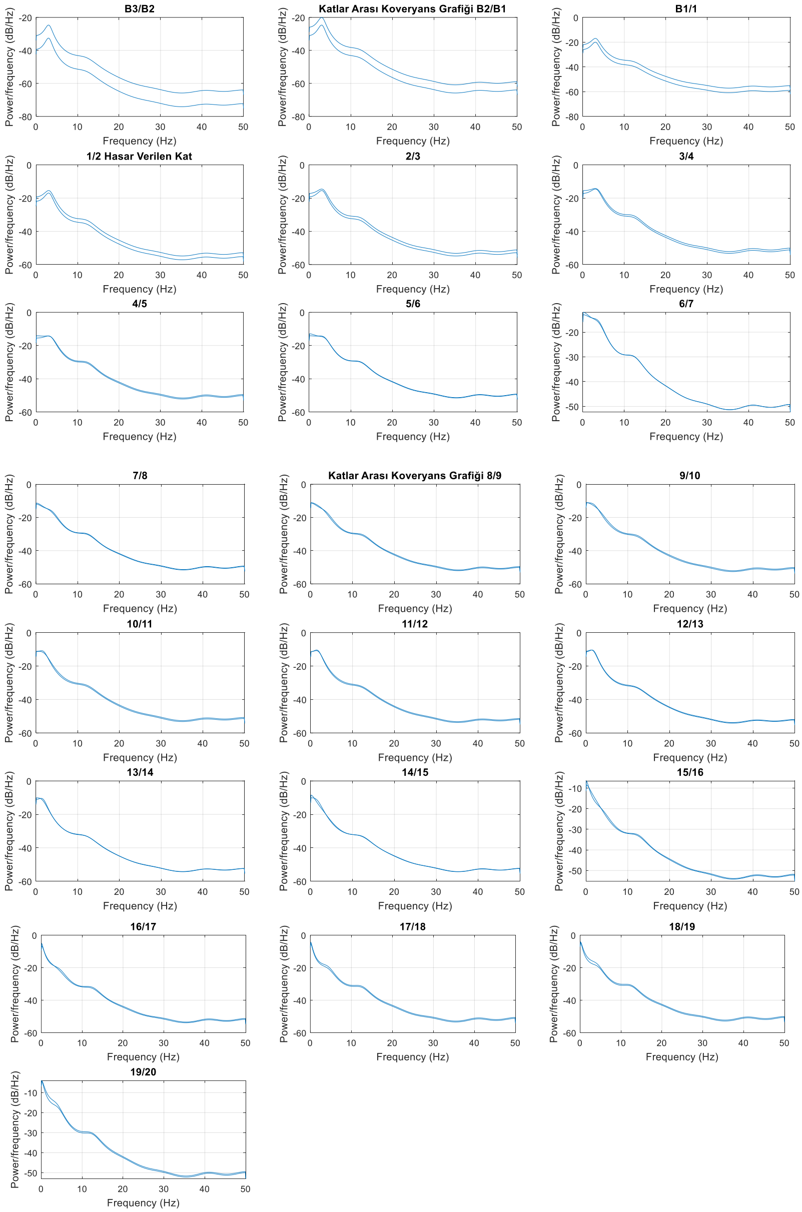

Figure 6. Spectral Ratio Charts Between Floors

Db4 was used for the wavelet transform and no visible difference was observed in the first-order frequency range for damaged and undamaged states, as can be seen in Fig. 7. However, when covariance, correlation, and correlation-like statistical operations are performed between the transformations, the differences as shown in Figures 7, 8, and 9 appear.

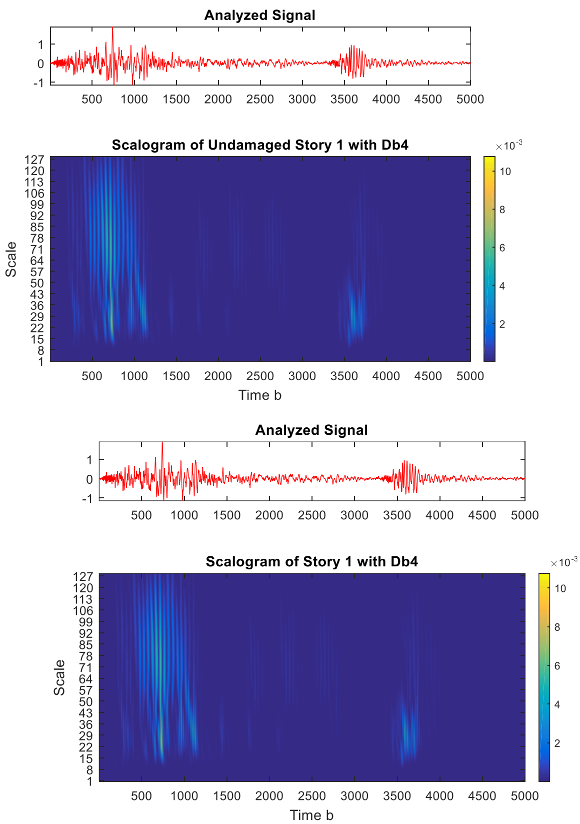

Figure 7. Wavelet Transforms of 1st Floor Damaged and Undamaged Cases

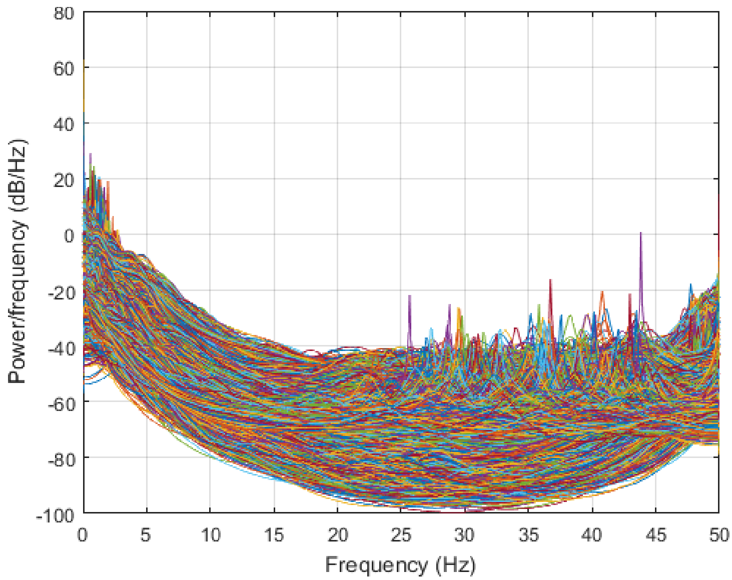

Figure 8. Cross-Correlation of Wavelet Transforms of 1st Floor Damaged and Undamaged Cases

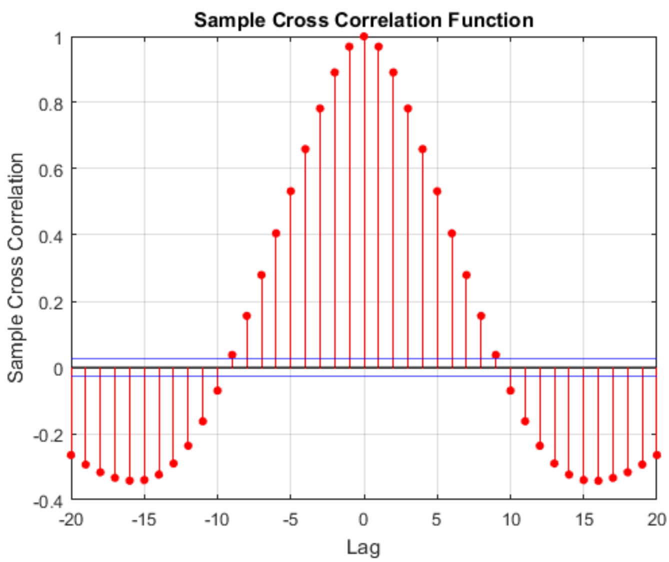

Figure 9. Correlation of Wavelet Transforms of 1st Floor Damaged and Undamaged Cases

In addition, when wavelet packet transformation applied in damaged and undamaged acceleration values are compared, it is observed that low-frequency modal parameters are obtained, and the frequency order changes at 3% level in Fig. 10.

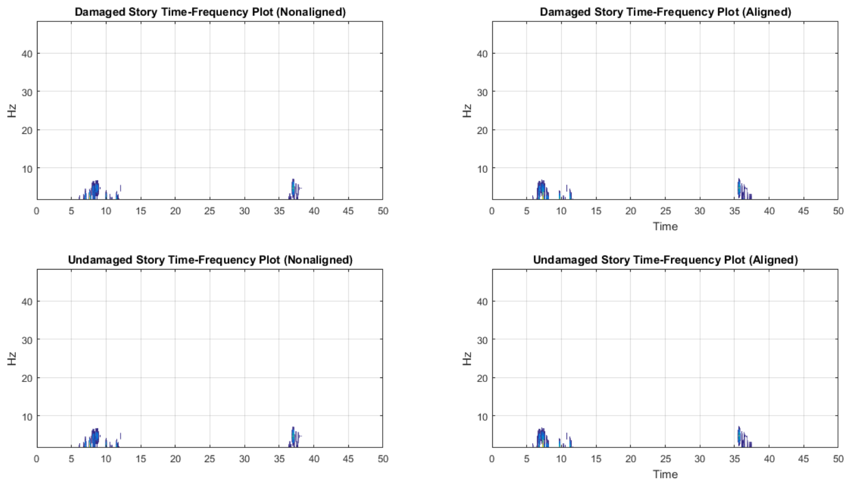

Figure 10. Wavelet Packet Transformation of Damaged and Undamaged Data on 1st Floor

When the signals obtained from the speed with Hilbert Huang transformation are examined, the frequency in which the frequency order is changed is observed in Figures 11 and 12.

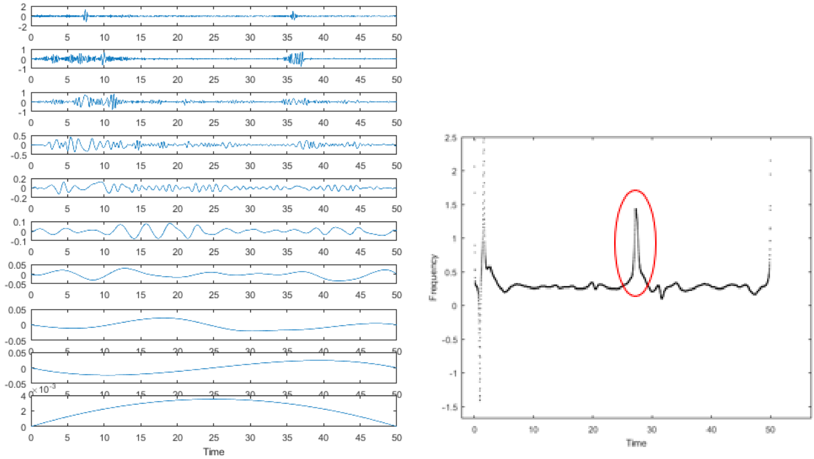

Figure 11. Hilbert Huang Transformation of Damaged Data of 1st Floor

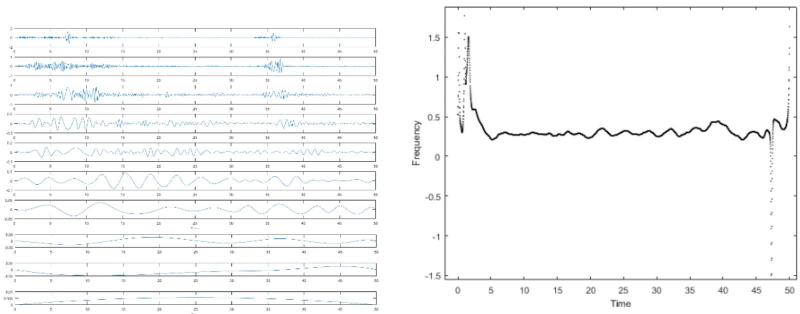

Figure 12. Hilbert Huang Transformation of Undamaged Data of 1st Floor

When the Hilbert Huang transformation is performed after the low pass filter operation, the frequency change can be seen more clearly as in Fig. 13.

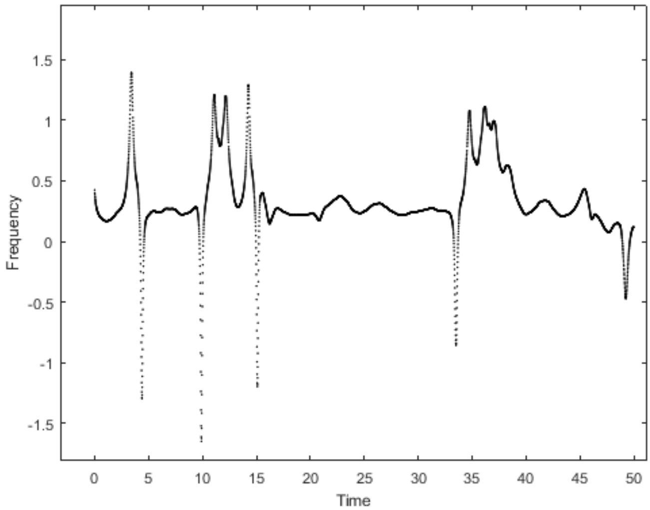

Figure 13. Hilbert Huang Transformation of Filtered Damaged Data of 1st Floor

4. Conclusions

As shown in this study, the Fourier transforms with the filters succeed in catching modal parameters in the frequency domain. Still, depending on the time, it is not possible to obtain information about the frequency change. In the wavelet and Hilbert Huang transformations, time-dependent changes of the building frequencies can be observed, and it’s shown that there are some differences between damaged and undamaged structures. The spectral ratios between the floors can determine the damaged story.

To ensure earthquake safety in our cities where multi-story buildings grow rapidly, it is possible to monitor their health instantly by controlling the similarity of the modal parameters predicted in the projects of these constructions with the modal parameters in the constructed structure.

As a result, it is possible to detect the frequency losses and the element that is damaged at the advanced level from acceleration, speed, displacement, and energy values. Using the spectral ratios obtained from the speed, the damage on the layer basis is given at a better level. Hilbert transforms and wavelet transforms give close and consistent results to the frequency domain obtained by the finite element method in the speed domain. For this reason, more efficient results can be obtained by taking speed and energy parameters as the basis for damage diagnosis.

5. Data Availability Statement

Some or all data, models, or codes that support the findings of this study are available from the corresponding author upon reasonable request.

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CHAPTER III

A View of Soil-Structure Interaction

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Introduction

The seismic response of a structure is mainly governed by the characteristics of the earthquake input motions. The characteristics of the input motions reaching to the structures are controlled by the fault mechanism, travel path and local site conditions, as seen in Figure 1. It is generally accepted that earthquake waves propagate within the horizontally layered soil layers in vertical direction. Without the structure on top of the soil or rock surface, the motion recorded on the soil deposit is called

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“free-field input motion” and the motion recorded on the rock is named “outcrop input motion”.

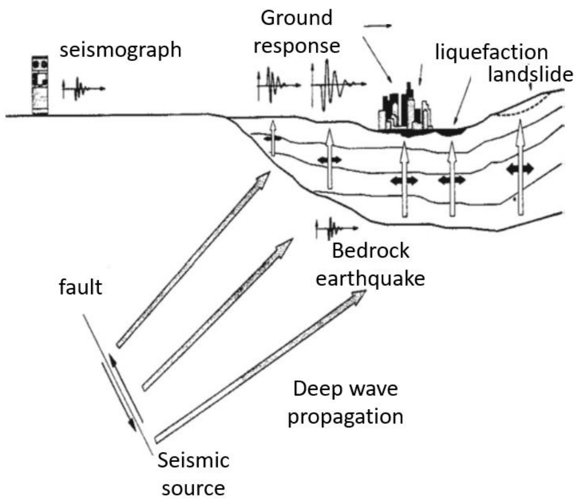

Figure 1: Illustration of the fault mechanism, travel path and local site conditions impacting the characteristics of the earthquake input motions.

As the structures are built on top of soil deposits, the soil and structure are inevitably interact in static or dynamic conditions. In the static condition, the bearing capacity and settlement potential are two main parameters of the soil layers underneath the building, which need to be satisfied. Therefore, the interaction between soil and structure matters only with respect to the aforementioned soil characteristics. However, in the dynamic condition, the interaction becomes especially critical. Since the movements of the soil and building, therefore their interaction, change constantly during the seismic excitation. The interaction is mainly the concern of two civil engineering disciplines: structural engineering and geotechnical engineering. This book section aims to give a brief view of soil-structure interaction.

Soil-structure interaction

Soil-structure interaction (SSI) can be interpreted as the process where the motion of the structures is affected by the response of the soil and the soil motion is influenced by the response of the structures (S. H. R. Tabatabaiefar, Fatahi, & Samali, 2013). When the base of the structures is modelled as fixed in which the foundation and soil support are considered as un-deformable, SSI effects are neglected. Conversely, if the structures are analyzed as flexible base in which deformations of foundation and supported soil are allowed, the modelling take SSI effects into consideration (NEHRP, 2012). An exemplary model that includes both soil and structure is demonstrated in Figure 2.

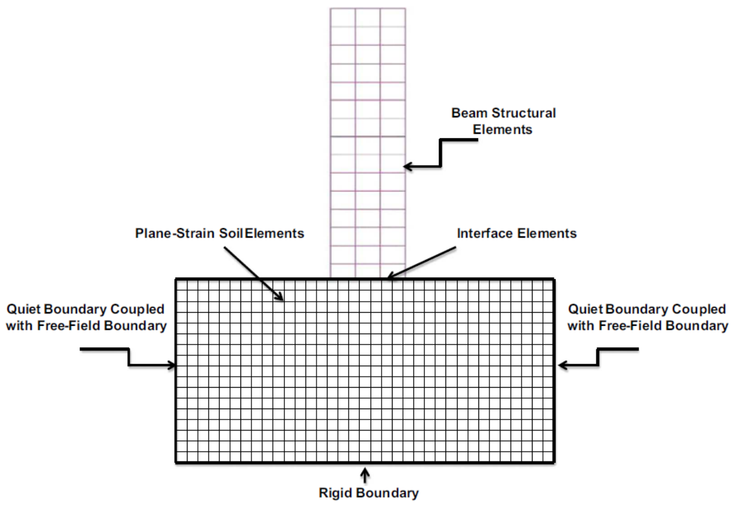

Figure 2: Example of a modelling system consisting of soil and structure, together with required boundary conditions (from H. R. Tabatabaiefar and Fatahi (2014)).

Since the performance-based seismic design of the structures in the seismic zones has gained popularity, the importance of the SSI effects on the seismic behavior of the structures has recognized, as well. The leading result of the SSI consideration in the seismic analysis is that the excitation at the foundation level of the structures is not identical to the excitation experienced by the soil in the same level of the free field. The first reason for this distinction is, partially, because of the refracting of seismic waves by the foundation whose stiffness and geometry differentiate its deformation from the one occurred relatively at the soft soil. Secondly, it is due to the fact that inertia forces of the structures during excitation result in deformations and displacements occurred in the soil by means of foundation. The first and second reasons refer to “kinematic interaction” and “inertial interaction”, respectively (Roesset, 2013).

There are generally two methods to analyze the seismic performance of the structures by considering SSI. (1) Direct Approach in which the soil and structure system are modelled and solved together in a single step. The structure is formed by the unity of linear members and finite elements, and the soil is divided into the parts. (2) Substructure or Three Step Approach where three steps are considered to analyze soil-structure system. These are assigning the seismic excitations of the foundation in the free-field condition, computing the dynamic stiffness of foundation and attaching the stiffness of foundation to the structure and inputting the motions obtained in the first step to analyze seismic performance of the structure (Roesset, 2013).

The direct approach, called the "complete solution" by Seed, Whitman and Lysmer (1977), can be ideal if nonlinear time-domain analyses were performed on detailed three-dimensional models of both the soil and the structure. This approach will incorporate with appropriate nonlinear material behavior for the soil and accounting for all types of waves. However, in practice, such analyses were typically carried out using two-dimensional plane strain models or pseudo three-dimensional models. These analyses were performed in the frequency domain, utilizing equivalent soil properties derived from iterative linear analyses, which adjusted the shear modulus and damping based on the characteristic shear strains from the previous cycle. The analyses also assumed uniform motion across all points along the model's bottom boundary, simulating vertically propagating shear waves. Structural models were generally simpler, sometimes consisting only of blocks of finite elements.

Performance evaluation

In order to analyze soil-structure system under seismic motion, there are two prevailing approaches that soil can be simulated, namely equivalent linear and complete nonlinear methods. The equivalent linear method is not enough to consider nonlinearity of dynamic soil-structure interaction, directly, since it presumes that the soil and structural elements are regarded as linear during solution stages in frequency domain. H. R. Tabatabaiefar and Fatahi (2014) describe process of equivalent linear method as follows.

- It is based on linear analysis, in which initial damping and shear modulus values are included.
- Then, maximum shear strain is recorded for each element and utilized to determine corresponding new damping ratio and shear modulus values in the backbone curves where damping ratio and secant modulus (G/G_{max}) are related to shear strain.(In this

stage, some empirical scaling factors are followed to relate these strains to the model strains.)

- Following that, new values of damping ratio and shear modulus are used in the next stage of the analysis.
- This process is retreated until the strain dependent values and structural response are unchanged.
- At the end, values of damping ratio and shear modulus have to be compatible with the strain value at the element. Rayleigh damping can be included in order to consider the energy dissipated by the seismic actions of soil-structure interaction system. To presume some influences of nonlinearity, the secant modulus and damping values are taken into consideration with their mean values.

Fully nonlinear analysis is accepted as the most suitable one to model and capture soil-structure interaction under seismic excitation because of its listed functions (H. R. Tabatabaiefar & Fatahi, 2014; S. H. R. Tabatabaiefar et al., 2013):

- Following any prescribed constitutive model including nonlinearity.
- Irrecoverable displacement and other permanent actions are automatically considered.
- Shear modulus in small strain range with damping value can be accurately computed in the model.

- In the dynamic analysis, the amount of energy dissipated by the soil deposit can be reproduced in the same magnitude and form by the damping of the system in the numerical modelling.
- In the soil and rock, the natural damping is hysteretic where secant modulus and damping functions are accounted for in the numerical modelling.
- P and S waves (primary and secondary or compressive and shear waves, respectively) are considered in one simulation so that the real response of the soil-structure interaction system can be captured.
- Structural elements are assumed as isotropic and linear elastic including no failure limit for linear elastic analysis or elasto-plastic with moment plasticity limit for inelastic analysis of the structures.

As highlighted by Hashash and Park (2002), for equivalent linear analysis, viscous damping is used to compute damping at small strain level by employing Rayleigh damping which is mass and stiffness matrix dependent, considering first natural mode. They also conclude that hysteretic damping in fully nonlinear analysis in time domain is able to represent damping larger than 10^{-4} (%) –10^{-2} (%), varying with material properties. Hence, nonlinear dynamic SSI analysis at small strains cannot capture damping accurately and may lead to underestimate the responses at that strain level. To prevent this, in the early stage of the cyclic loadings, viscous damping can be introduced. Hardin model (Hardin & Drnevich, 1972) which is one of several tangent modulus functions has been used by many researchers (Fatahi & Tabatabaiefar, 2014; H. R. Tabatabaiefar & Fatahi, 2014; S. H. R. Tabatabaiefar et al., 2013) to describe hysteretic damping in which shear modulus ratio, damping ratio and cyclic shear strain with numerical fitting parameters can be represented. Depending on the soil type (sand, clay etc.), numerical fitting parameters can generate backbone curves during dynamic analysis of soil-structure interaction systems. The formula for Hardin model is:

$$\mathbf{M_s} = \frac{\mathbf{1}}{\mathbf{1} + {}^{\boldsymbol{\gamma}}/_{\boldsymbol{\gamma}_{\mathbf{ref}}}}$$

M_s is the secant modulus (G/G_{max}), γ is the cyclic shear strain and γ_{ref} is Hardin/Drnevich constant as the numerical fitting constant which is considered as 0.006 for sandy soils (Seed, Wong, Idriss, & Tokimatsu, 1986) and 0.234 for clayey soils (Sun, Golesorkhi, & Seed, 1988).

Another study has done by Amorosi et al. (2010) to reveal the differences between the results of site response in the equivalent visco-elastic analysis, which is one-dimensional frequency domain approach, and those of site response in time domain 2D Finite Element (FE) analysis. For equivalent visco-elastic analysis EERA and for finite element analysis SWANDYNE are used. It is important to say that in the study, the site response results of EERA have been regarded as the target for the FE analyses. To make the results of these two approaches (EERA and SWANDYNE) comparable, new calibration method has been developed for the linear elastic and viscous parameters which gives good matching between results of EERA and SWANDYNE in terms of peak ground acceleration versus depth and versus f_p/f_1 (f_p is the predominant frequency of the input motion and f_1 is the fundamental frequency of the soil deposit) and the use of advance constitutive modelling has been avoided in FE analyses. These parameters also enable the practitioner to introduce same constitutive model for two different site response analysis methods. They investigate the effect of nonlinearity with regard to hysteretic damping and viscous damping, boundary conditions, spatial discretization and time integration parameters.

For the influence of soil plasticity, non-associated visco-elasto-plastic constitutive model has been adapted in SWANDYE the damping values are varied based on EERA damping values with each layer. It is found that none of the response spectra of the three attempts has not been able to have a good match with EERA results at upper layers their peak values are less than EERA ones even if their forms are identical. This can be attributable to the hysteretic damping and the high value of strains at the uppermost part of the soil deposit. Because of these reasons, EERA results cannot be the right target to achieve in FE analyses under strong input motion as visco-elastic constitutive models cannot capture irreversible strains in relation to evolution of effective stress due to built-up excess pore pressures. Hence, using time-domain FE model to analyze the seismic behavior of the site can attain non-linearity of the soil more realistically.

Mohr-Coulomb model has generally be adapted as a constitutive model to the soil-structure system to simulate nonlinear behavior of the soil deposit under dynamic loading. This constitutive model is elastic-perfectly plastic method used by many researchers (as Conniff and Kiousis (2007); Rayhani and El Naggar (2008)) in the soil-structure interaction models.

Simplification of SSI

The detailed modeling of the foundation and the surrounding soil, which includes all material nonlinearities (such as irreversible soil behavior and sliding along the interface) and geometric complexities (like foundation uplift), is both intricate and computationally demanding. As a result, engineering practice often favors the development of simplified models to describe the foundation-soil system. These simplified models can be integrated into global models for the superstructure and are designed to capture the non-linear effects at the foundation level that are crucial for the overall design. This discussion focuses on shallow foundations, which can typically be treated as rigid relative to the soil beneath them. There is extensive literature on simplified models for these cases, which can generally be divided into two categories: (a) "macroelement" (or force-resultant) models, and (b) models based on the Winkler decoupling hypothesis, as demonstrated in Figure 3.

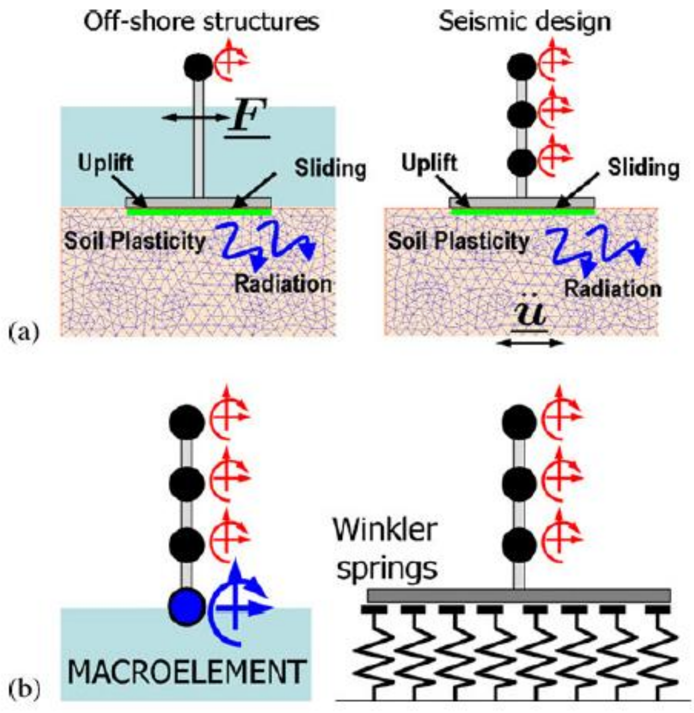

Figure 3: (a) Soil-structure interaction scenarious for shallow foundation systems (b) simplifications of such cases (taken from Chatzigogos, Figini, Pecker, and Salençon (2011)).

In the first category, the footing and the soil are treated as a single "macroelement," and a model with six degrees of freedom (for 3D cases) or three degrees of freedom (for 2D cases) is created to describe the force-displacement behavior of a specific point on the footing, usually its center, in vertical, horizontal, and rotational directions. In the second category, the soil is represented by a series of decoupled horizontal and vertical springs, each with an appropriate constitutive law (such as elastoplastic or contact-breaking). The decoupling hypothesis is advantageous because it simplifies the process of integrating the local spring responses to obtain the overall footing response. However, it has limitations, including challenges in calibrating the model parameters and difficulties in describing the interactions between vertical/rotational and horizontal degrees of freedom of the footing.

Studies considering SSI

H. R. Tabatabaiefar and Fatahi (2014) have studied soil-structure interation. They considered 5, 10 and 15 storey buildings. The results indicated that the seismic responses of structures resting on firm soil having shear wave velocity greater than 600 m/s are not affected by SSI. More clearly, considering the building with fixed base or flexible base in the model does not influence the behavior of the structures under dynamic condition since the soil medium have shear wave velocity more than 600 m/s. In contrast to this, having soil with shear wave velocity less than 600 m/s supporting the structures may necessitate taking SSI effects into consideration in the modelling of soil-structure system with the view to assessing the seismic behavior of the structures, rationally.

It has been proved by the study of S. H. R. Tabatabaiefar et al. (2013), carrying out nonlinear dynamic analysis of the structure resting on soil classes C, D and E by considering SSI in FLAC 2D (finite difference method), that the less the soil properties (e.g. shear wave velocity, stiffness), the more SSI effects are pronounced (soil classes according to shear wave velocity are given in Table 1). They proved that the decreases in the dynamic features of the soil lead to decrease in the shear forces, and increase in the natural period of the structures. They conclude that the seismic performance level of the structure on the soil class C does not change distinctively while there has been crucial change in the seismic performance of the structure resting on soil classes D and E.

Table: Soil classes according to shear wave velocity given by NEHRP

Soil class	A	B	C	D	E
$V_{s,\,30}$ (m/s)	>1500	760-1500	360-760	180-360	<180

As the height of the building increases, so thus the natural period of the building, inter-storey drift ratios are also risen, too. This case is particularly true when the building is supported by soft soils (especially in soil class E). Because the soft soils owns higher spectral values at the longer periods where the building period fits. It can be said that neglecting SSI in the modelling of structures resting on firm soil can be beneficial, whereas ignoring SSI in the modelling of the structures resting on soft soil can be detrimental. Additionally, it is generally introduced by modern seismic codes that SSI is beneficial in relation to reducing force demand of the structures.

In addition, the study by El Ganainy and El Naggar (2009) found that, by considering SSI, shear force and moment demands of the structures on the firm soil decrease, while both of them increase in the condition where the structures are rested on the soft soil. They also investigate SSI effects in relation to the number of underground stories. The results show that SSI effects has inverse relationship with number of underground stories since SSI effects decrease, as a result, shear forces and inter-storey drift ratios get closer to those of fixed ones, with increasing number of underground stories. This can be due to the fact that rigidity of basement walls and lateral resisting system of the underground storey slabs are comprised of rigid box embedded in the soil and, consequently, fix the structures.

Conclusion

The continuous functionality of the buildings throughout its design period is essential for social and economic reasons and for the safety of dwellers. Such design purpose should be met under the static and dynamic conditions. In order to deliver this, the buildings should be designed in accordance with the modern seismic desing codes. In addition, the soils supporting the buildings should be well featured so that they can safely carry the building loads with acceptable settlement. Moreover, the interaction of the soil with the buildings should be clearly identified, especially in the dynamic condition.

This section of the book highlights the several aspects of the soil-structure interaction. In particular, following points are underlined:

- There are generally two main approaches to consider soil-structure interaction: (i) direct approach and (ii) substructure approach,
- The soil can be simulated by considering equivalent linear or nonlinear methods,
- Boundary conditions should be applied properly to replicate the real conditions as possible,
- Simplification of the soil-structure problem is very practical and reasonable
- The soil class seems to have critical role in the soil-structure problems, where as the soil gets softer, the

consideration of soil-structure interaction in the dynamic analysis becomes more critical.

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CHAPTER IV

Exploring the Potential of Apricot Kernel Shell Powder in Gypsum-Based Mixtures

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Introduction

Apricot (*Prunus armeniaca L.*) is a fruit belonging to the Prunus genus and the Armeniaca subsection of the Rosaceae family. The native regions of apricot are Central Asia, Western China, and the Iran-Caucasus region. It is an important fruit grown in many countries, including those in the Mediterranean region (Koç, 2023). Türkiye ranks among the top countries in the world in terms of

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apricot sapling planting area and production. Due to its high nutritional value, apricot has become an increasingly popular product for consumption (Taşkın, 2024). Apricots can be used for canning, fresh consumption, drying, freezing, and industrial purposes. Although a large portion of apricot production worldwide is consumed fresh, in Türkiye, it is primarily processed as dried apricot. Dried apricots can be stored easily and for long periods due to their low moisture content (Koç, 2023). In the spring months, the occurrence of frost due to a drop in temperature during the apricot flowering period significantly affects apricot production, leading to fluctuations in yield from year to year (Taşkın, 2024). According to 2021 data, Türkiye is the country with the largest apricot cultivation area in the world, covering 135,000 hectares. It is also the leader in apricot production with 824,000 tons and apricot exports with 97,000 tons. Türkiye holds 22.4% of the global apricot production, 27.5% of fresh apricot exports, and 53.3% of dried apricot exports. The majority of production is concentrated in the provinces of Malatya, Mersin, Elazığ, Kahramanmaraş, and Iğdır (Hasdemir, 2023).

Apricots are widely recommended for human consumption due to their numerous health benefits, which include being rich in fiber, minerals, and various vitamins such as Vitamin A, Vitamin C, Vitamin K, thiamine, riboflavin, niacin, and pantothenic acid. In addition, they are packed with antioxidants, primarily from phenolic and carotenoid compounds (Aljoumaa et al., 2017). Apricots are reported to have positive effects in the treatment of conditions such as heart failure, kidney diseases, hepatitis, and cirrhosis due to their low sodium and high potassium content. One of the most important compounds in dried apricots for nutrition and health is dietary fiber. Dried apricots are rich in dietary fiber, which consists of compounds such as polysaccharides and lignin that cannot be hydrolyzed by enzymes secreted in the digestive system. Dietary fiber helps reduce the risk of diseases like constipation, irritable bowel syndrome, appendicitis, hemorrhoids, dental diseases, obesity, diabetes, coronary heart disease, and colon cancer, while promoting regular bowel function (Alan et al., 2013).

After the fleshy part of the apricot fruit is separated, the remaining part is the seed. This seed constitutes approximately 10% of the total weight of the apricot fruit (Soleimani & Kaghazchi, 2008). The flesh-to-seed ratio of apricot varieties in Türkiye can range from 11/1 to 15/1 (Güner et al., 1999). The kernels are utilized in cosmetic production because they contain dietary proteins, fiber, oils, and benzaldehyde, all of which demonstrate antioxidant and antimicrobial properties (Yiğit et al., 2009). In order for apricot kernels to be ready for use, they must be cracked open to extract the inner parts (the almonds) (Güner et al., 1999). Unlike apricot almonds, the broken shells remain as waste. These are often utilized as fuel in various places (Aljoumaa et al., 2017). Additionally, it has been observed in the literature that composite construction materials are produced by incorporating them into cement-based products or mixing them with polymers (Çelik et al., 2021; Yildiz et al., 2012). Many studies have focused on the production of activated carbon from waste apricot kernel shells (Fadhil, 2017; Janković et al., 2019; Shaikhiev et al., 2022; Vashchynskyi et al., 2023).

Ground apricot kernel shells are successfully utilized in tank factories and industrial facilities requiring metal cleaning. This product is also employed in the maintenance and cleaning of aircraft engines as well as metals such as copper and aluminum in the United States and many European countries. Currently, surface cleaning processes are carried out using silica sand, steel grit, and mineral slags. Ground apricot kernel shells offer a significant advantage over previously used chemical-based abrasive products, being entirely natural (Çemrek, 2011). However, the high volume of apricot production in Türkiye results in substantial amounts of shell waste, even though the shell constitutes only about 10% of the apricot fruit by weight. Developing new applications for utilizing this agricultural waste is of significant economic and ecological importance.

In this study, apricot kernel shells (AKS) were processed through grinding to obtain a powdered form. The resulting powdered material was utilized in varying proportions in gypsum-based mixtures, a widely used construction material. To examine the impact of this waste material on gypsum-based mixtures, several physical and mechanical tests were conducted.

Materials

The gypsum used in this study was obtained as a commercial product conforming to the TS EN 13279-1 standard. The AKS used in the study were procured in pre-ground form, with a fineness passing through a 60-mesh sieve (<250 μm). The technical properties of the gypsum used are provided in Table 1, while its chemical and physical analysis is detailed in Table 2. The chemical and elemental analyses of the AKS powder are presented in Tables

3 and 4, respectively. Images of the materials used are shown in Figure 1.

Table 1: Technical properties of gypsum

Property	**Value**
Initial setting time (min)	> 8
Final setting time (min)	≈ 30
Minimum compressive strength (40x40 mm) (MPa)	10
Minimum flexural strength (40x40x160 mm) (MPa)	4.5
Grains under 200 μm (min)	%99.5
Grains under 100 μm (min)	%95.0
Loose unit weight (g/cm^3)	0.75 – 0.80
Dry unit weight (g/cm^3)	1.05 – 1.10

Table 2: Chemical and physical analysis of gypsum

Property	**Value**
SiO_2	3.16%
Al_2O_3	0.44%
Fe_2O_3	0.37%
CaO	40.71%
MgO	0.61%
SO_3	51.79%
Na_2O+K_2O	0.26%
Loss on ignition	18.84%
Density	2.29 g/cm^3
Fineness	500 m^2/kg

Table 3: Chemical analysis of apricot kernel shell powder

Chemical analysis	**Quantity (%)**
Extractive substance	5.074
Hemicellulose	30.16
Cellulose	33.87
Lignin	30.15
Ash	0.750
Holocellulose	64.03

Reference: (Ceylan et al., 2020)

Table 4: Elemental analysis of apricot kernel shell powder

Elemental analysis	**Quantity (%)**
C	47.50
H	5.414
N	0.200
O	46.49
S	-
H/C	0.11
H/N	0.00

Reference: (Ceylan et al., 2020)

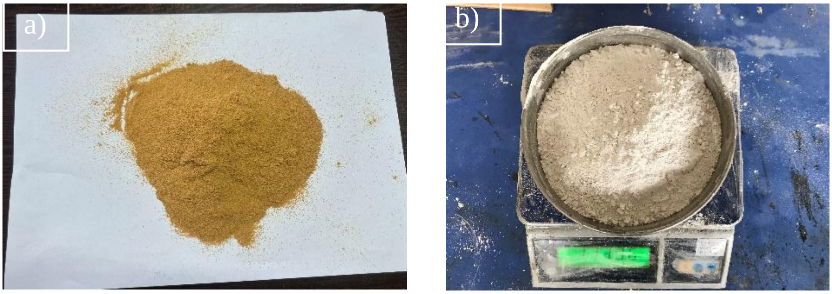


Figure 1: Images of a) Gypsum b) Apricot kernel shell powder

Methods

The mixture proportions used in the study are presented in Table 5.

Table 5: Mixture proportions (g)

Code	Gypsum	Water	Apricot kernel shell powder
R	1200	780.0	-
K3	1175.8	764.3	15.6
K6	1151.6	748.6	31.2
K9	1127.5	732.8	46.8
K12	1103.3	717.1	62.4
K15	1079.1	701.4	78.0
K20	1038.8	675.2	104.0

Ground AKSs were utilized as a replacement for gypsum at volumetric proportions of 3%, 6%, 9%, 12%, 15%, and 20%.

Initially, gypsum and AKS powder were mixed in a laboratory-type cement mixer for 30 seconds to achieve homogeneity. Subsequently, water was gradually added, and the mixture was stirred at low speed for 30 seconds followed by high speed for 60 seconds. The prepared mixtures were then promptly poured into molds with dimensions of 40x40x160 mm and subjected to vibration for compaction.

After 24 hours, the samples were demolded and cured under laboratory conditions. Following a 7-day curing period, the specimens were placed in an oven at 65 °C for 48 hours to remove moisture. At the end of this process, the tests outlined in Table 6 were conducted on the samples. Visual representations of these tests are shown in Figure 2.

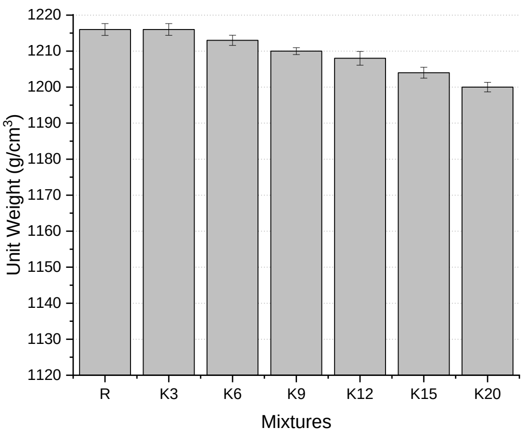
Table 6: The tests conducted in the study and their corresponding standards

Test	**Standard**
Unit weight	ASTM C 138
Ultrasonic pulse velocity (UPV)	ASTM C 597
Water absorption	ASTM C 20
Apparent porosity	ASTM C 20
Capillary water absorption	TS EN 480-5
Flexural strength	TS EN 196-1
Compressive strength	TS EN 196-1

Figure 2: Visuals of experimental studies

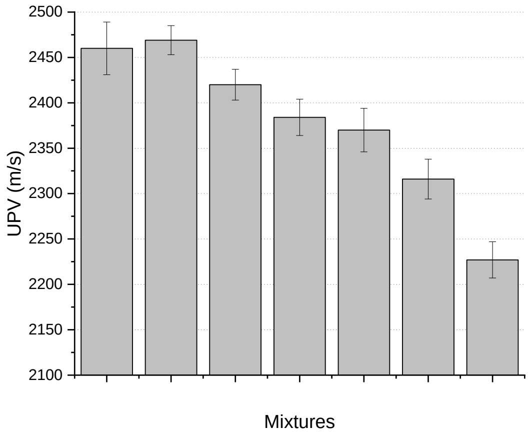
Results and Discussion

Graphic 1 presents the results of the unit weight test.



Graphic 1: Unit weight results

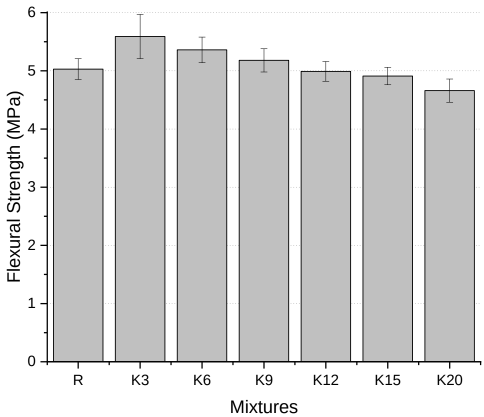
The unit weight values range between 1200 and 1216 kg/m³. As seen from the results, the addition of AKS powder to the mixture did not exhibit a significant effect on the unit weight. However, it can be stated that the use of AKS powder caused a slight decrease in the results. The total variation between R and K20 is only about 1.3%. A downward trend in the results can be observed in Graphic 1. This is attributed to the lower unit weight of AKS powder compared to gypsum.



Graphic 2: UPV results

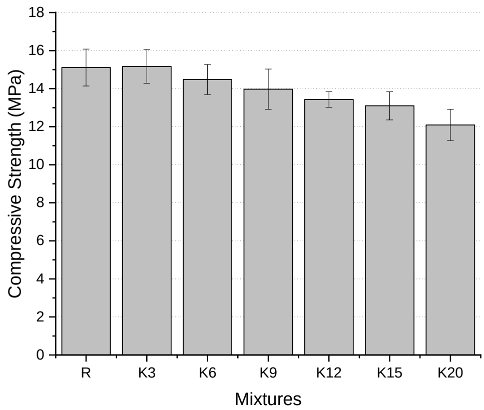
Graph 2 illustrates the UPV results, which range between 2227 and 2469 m/s. The reference sample recorded a UPV value of 2460 m/s. Upon examining the results, it is observed that the use of 3% AKS powder slightly increased the UPV value, whereas higher percentages led to a decrease in these values. Specifically, the incorporation of 3% AKS powder resulted in a marginal increase of 0.3% in the UPV value, while increasing the usage to 6% caused a 1.6% decrease. For the K20 sample, the UPV value dropped by 9.5%.

The magnitude of the UPV value is generally associated with the material's internal structure being dense and void-free. Conversely, a lower UPV value indicates a more porous and loose internal structure. These findings can also be correlated with the material's mechanical properties (Benaicha et al., 2015). However, it is essential to note that the observed changes in UPV values in this study are minimal.



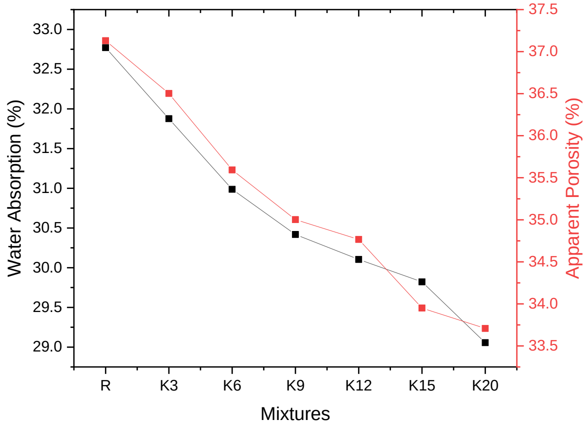
Graphic 3: Flexural strength

Graphic 3 shows the flexural strength test results. The flexural strength values range between 4.66 and 5.59 MPa. The reference sample exhibits a flexural strength of 5.03 MPa. The incorporation of 3% AKS powder increased this value to 5.59 MPa, representing an approximately 11.1% improvement. At the same time, higher results were obtained compared to the reference sample at the 6% and 9% usage rates. The increase caused by these proportions is 6.5% and 3%, respectively. However, the use of AKS powder at proportions higher than 9% resulted in a decrease in flexural strength. The lowest value was recorded for the K20 sample, which was approximately 7.4% lower than the reference sample.



Graphic 4: Compressive strength

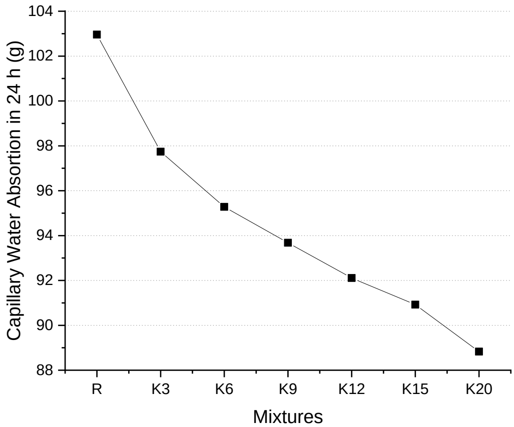
Graphic 4 shows the results of the compressive strength test. The compressive strength values range between 12.09 and 15.17 MPa. The reference sample has a compressive strength of 15.11 MPa. As seen, the use of 3% AKS powder has caused a very slight increase in compressive strength, similar to the flexural strength values (only 0.4%). After this proportion, a decrease is observed again. The K20 sample exhibited the lowest compressive strength value, with a reduction of approximately 20% compared to the reference sample. As seen from both mechanical properties, the changes resulting from the use of small amounts of AKS powder are not significant.



Graphic 5: Water absorption and apparent porosity results

Graphic 5 illustrates the results of the water absorption and apparent porosity tests. The water absorption values of the samples range between 29.05% and 32.77%, while the porosity values vary from 33.71% to 37.13%. The highest values for both tests were obtained from the reference sample. It was observed that the inclusion of AKS powder and the increase in its usage proportion led to a gradual decrease in these values.

Gypsum is inherently a porous material. It is believed that the AKS powder, which does not participate in chemical reactions, acts as a filler within the voids of the material. Additionally, the low water absorption capacity of AKS particles is thought to contribute to the observed reduction in water absorption values (Ishaq et al., 2024).



Graphic 6: Capillary water absorption results

Graphic 6 shows the amount of water absorbed by the samples through capillary action after 24 hours. The capillary water absorption values range between 88.8 g and 103 g at the end of 24 hours. The highest capillary water absorption was recorded for the reference sample. An increase in the proportion of AKS powder resulted in a reduction in capillary water absorption values. This is believed to be due to the low water absorption capacity of the AKS powder and its role in occupying voids within the material.

Conclusions

This study investigated the effects of incorporating apricot kernel shell (AKS) powder into gypsum-based composites. Based on the experimental results, the following conclusions can be drawn:

1. The addition of AKS powder caused a slight decrease in unit weight, with values ranging between 1200 and 1216 kg/m³. The total variation was minimal (approximately 1.3%), and the reduction

is attributed to the lower density of AKS powder compared to gypsum.

2. The UPV values decreased as the AKS powder content increased, except for the 3% mixture, which showed a negligible improvement (0.3%). Higher proportions of AKS powder resulted in a reduction of up to 9.5%. The UPV results indicate that AKS powder contributes to a more porous internal structure, though the changes are minor.

3. 3% addition of AKS powder improved flexural strength by approximately 11.1%, with moderate improvements at 6% and 9% usage rates. However, higher proportions caused a reduction, with the K20 sample exhibiting the lowest value, 7.4% below the reference. Similar to flexural strength, a slight increase (0.4%) was observed with 3% AKS powder. However, higher additions led to a decline, with the K20 sample showing a 20% reduction compared to the reference.

4. Both water absorption and apparent porosity values decreased with the inclusion of AKS powder. This is likely due to the filler effect of AKS particles, which occupy voids and reduce the material's overall porosity and water absorption capacity.

5. A reduction in capillary water absorption was observed as AKS powder content increased. This can also be attributed to the low water absorption capacity of AKS powder and its role in filling void spaces within the composite.

In summary, the use of AKS powder as an additive in gypsum-based composites exhibited some beneficial effects at lower proportions, particularly in improving (or not changing) flexural strength and reducing water absorption and porosity. However, higher proportions of AKS powder generally led to a decline in mechanical properties. These findings suggest that small additions of AKS powder could be effectively utilized in gypsum-based composites, contributing to waste valorization while maintaining acceptable performance characteristics. Further research is recommended to optimize the usage proportions and investigate long-term durability.

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CHAPTER V

Transformation and Design of Village Structures on Site Post the Kahramanmaraş Earthquake

Başak ZENGIN[1]

INTRODUCTION

The 2023 earthquakes that struck Kahramanmaraş and the surrounding rural areas have led to significant destruction in residential areas. As in many regions of Turkey, there has been an urgent need for rapid reconstruction and on-site transformation projects in rural settlements following the earthquake. The structures in rural areas are often built with old, non-earthquake-resistant materials. This situation poses serious threats both to safety and economically.

Natural disasters, especially earthquakes, have an impact on the transformation and development of our cities. These effects on

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cities also indirectly affect the country as a whole. Especially in developing countries, losses caused by natural disasters are greater compared to developed countries. Therefore, studies are being conducted on disaster management strategies. As a result of these studies, at the current stage, measures to be taken and policies to be implemented before the occurrence of disasters are being aimed at minimizing the damages caused by the disaster (Düzgün and Yücemen, 2007). These people, who suddenly lost their homes, have expressed a desire to move to normal housing with better conditions than temporary shelters like tents or containers. However, the lack of sufficient housing stock to meet this demand has caused rents and house prices to rise rapidly (Özalp and Arslan, 2020). Urban transformation encompasses a holistic approach that includes the necessary environmental and physical improvements to solve urban problems, as well as the integration of the economic and social structure into these improvements. (Thomas, 2003). According to the United Nations' 2009 definition, urban resilience to disasters is the ability of settlements and communities to protect themselves against hazards, maintain the functioning of systems, quickly rebuild, and adapt to change, and to use these resources effectively. (Türkoğlu, 2014). While Elâzığ and its surroundings carry a high earthquake risk, past destructive earthquakes in the region and academic studies have predicted the potential damages. However, a large-scale transformation has not been carried out beforehand. Thousands of houses were demolished and transformed in a short period after the earthquake, demonstrating that urban transformation can be implemented quickly before a disaster. This situation emphasizes that transformations should be carried out before earthquakes occur (Ökde, and Ekinci, (2022)). In the case of Elâzığ city center, there are differing opinions among the public regarding the new TOKİ housing built after the Elâzığ earthquake on January 24, 2020. There are differing opinions in the public regarding how these earthquake-resistant houses respond to the city's lifestyle and daily needs. Especially the issue of how large families will live in the earthquake housing designed as 2+1 is being discussed among the public and the community (Sarışın and Akça, 2022,). The earthquake risk in Turkey, especially after the 1999 Izmit and Düzce earthquakes, has led to significant loss of life and property. The urban development of Gölcük has been shaped by post-earthquake replanning; due to the unsound soil structure of the coastal areas, new residential areas have been shifted to the southern city outskirts. The aim of this study is to analyze the pre- and post-earthquake processes in Gölcük in terms of urban spread and construction issues, both physically and socio-economically. The research examines historical changes, demographic structure, social life, and changes in economic values (Uzuner and Akıncıtürk 2020). In this study, the formation of cities after an earthquake and how urban culture is affected have been examined. In Kahramanmaraş, new settlement areas and previously earthquake-affected cities were observed, and the public's opinions were gathered. The study shows that new residential areas can have negative cultural and physical impacts, and that people want improvements in transportation and living standards. However, while some people support new residential areas to live in safe buildings, it has been observed that urban culture is weakening and neighborhood relationships are being lost. Additionally, it has been emphasized that new settlement areas should be built on solid ground, and the impact of the earthquake has increased due to the weakness of old buildings and illegal constructions. As a result, there has been a concern that moving to different residential areas could affect urban culture. (Speak 2024). It is observed that Chile has made regulations on land structures and taken more measures against tsunamis following major disasters like the 2010 Maule Earthquake. After the buildings were damaged by the earthquake and tsunami, expert teams were assigned for the reconstruction plans. These planning efforts aimed to create more livable cities. With the budget support package provided by the government for permanent housing, it is planned to build a 40 m^2 house for each family. The design team decided to build half a good house instead of a bad house with this budget. Therefore, the residences called "Villa Verde" have been designed in two phases. In the first phase, the foundation, infrastructure, roof, ground floor kitchen and bathroom, and an upper floor bedroom of each residence were constructed. The second stage will be carried out according to the users' budget, preferences, and desires (Kalkan et al., 2020).

Insite transformation means strengthening old structures on-site or completely rebuilding them. It is expected that such projects, especially in rural areas, will serve the purpose of creating safe and sustainable settlement areas by preserving the social structure, cultural heritage, and economic activities. However, on-site transformation projects have both advantages and various challenges. It is of great importance that the settlements built after the earthquake are designed with consideration for environmental sustainability, the quality of life of the local people, and the social structure.

In this context, the study will discuss the impact of on-site transformation in rural settlements, taking into account social, cultural, and structural elements. While cities are constantly evolving through change and development, projects that guide and implement these changes are utilized. The projects being implemented offer many advantages while enhancing both the urban landscape and the quality of communities. The on-site transformation project is among the projects carried out within this scope. The advantages provided by these projects can be listed as follows:

It provides earthquake safety and resilience.

Energy-efficient and environmentally friendly designs are made.

• It boosts economic vitality by increasing employment opportunities.

• Spaces that encourage socialization between communities are built.

• It creates a beautiful image in terms of aesthetics and architecture.

On-site transformation in housing is a process carried out with the aim of strengthening or renewing existing structures, especially after natural disasters such as earthquakes. There are successful examples of this in the world. Here are some of them:

In Turkey, on-site transformation practices in rural areas after earthquakes gained importance, especially following the 1999 Izmit and 2020 Elâzığ earthquakes. In rural areas, it is aimed to protect the social structure and create sustainable living spaces through on-site strengthening and renovation projects for risky buildings. Here are examples of on-site transformation in rural areas of Turkey after the earthquake:

Elâzığ- Sivrice Rural Transformation Projects

Following the 2020 Elâzığ earthquake, on-site transformation projects were initiated, especially in the villages located in the Sivrice district, aimed at repairing and strengthening the damaged houses. In this context, various financial supports have been provided to preserve the local structure and meet the needs of the people.

Malatya- Doğanyol and Pütürge

After the 2020 earthquake, on-site transformation works were carried out in the districts of Doğanyol and Pütürge. The aim is to strengthen or renew damaged and risky structures. Additionally, cooperation has been established with local administrations for the restructuring of social and living spaces in the villages.

Düzce- Rural Area Projects

Post-1999 Düzce earthquake, on-site transformation efforts in rural areas encompass the renovation and strengthening processes for old structures. Local governments, in cooperation with village heads, have ensured the construction of safer houses in the villages.

Bingöl- Rural Transformation Projects

In Bingöl, reconstruction processes in rural areas began after the earthquakes that occurred in 2010 and 2011. Various projects have been carried out for the on-site transformation of risky structures and the strengthening of social infrastructure.

These projects are important for reducing the damage caused by earthquakes in Turkey's rural areas, ensuring that people have safe living spaces, and protecting social structures. In-situ transformation stands out as a critical strategy for the reconstruction and sustainability of rural areas.

1. Kahramanmaraş Earthquake

On February 6, 2023, two major earthquakes centered in Kahramanmaraş and Elbistan occurred. There were two earthquakes centered in Kahramanmaraş-Pazarcık with a magnitude of Mw=7.7 at 04:17 local time (01:17 UTC) and in Kahramanmaraş-Elbistan with a magnitude of Mw=7.5 at 13:24 local time (11:24 UTC) on the same day. Additionally, a foreshock with a magnitude of Mw = 6.7 occurred in Gaziantep- Nurdağı at 04:28 local time (01:28 UTC), close to the first earthquake (AFAD, 2023-Figure 1). Frequent aftershocks have continued in the region. After the earthquake, on-site transformation has been initiated in both the cities and villages affected by the earthquake. After the earthquake, rural transformation projects are being implemented to strengthen local structures and reorganize social areas. These projects also include efforts to address the psychological and social difficulties the public faces after a disaster.

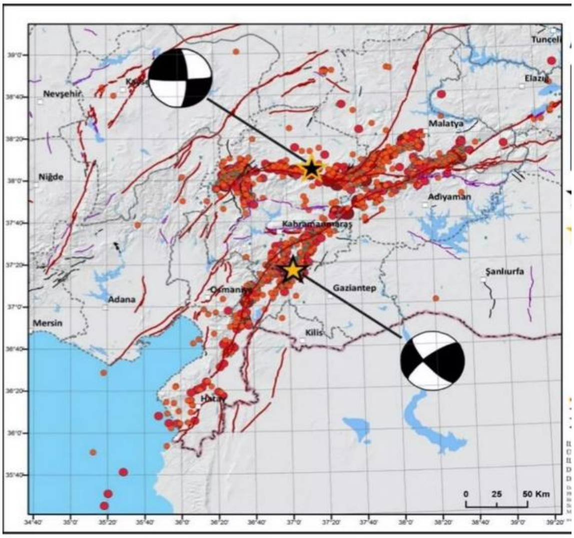

Figure 1. Earthquake locations

2. Buildings in the Area Outside the City

When rural structures are examined, they have varied according to the period in which they were built. The materials used also vary by region. In the old structures, there are adobe buildings, while in the newer ones, there are buildings designed with stone, brick, and tile. These structures are generally built without adhering to design principles. Structures are generally designed either without plaster or without any insulation (Figure 2).

Figure 2. Old damaged village house models

In village buildings, the pantry area and kitchen are important units, while the halls are designed to be spacious. Under the houses, there are usually either stables or storage areas (Figure 3).

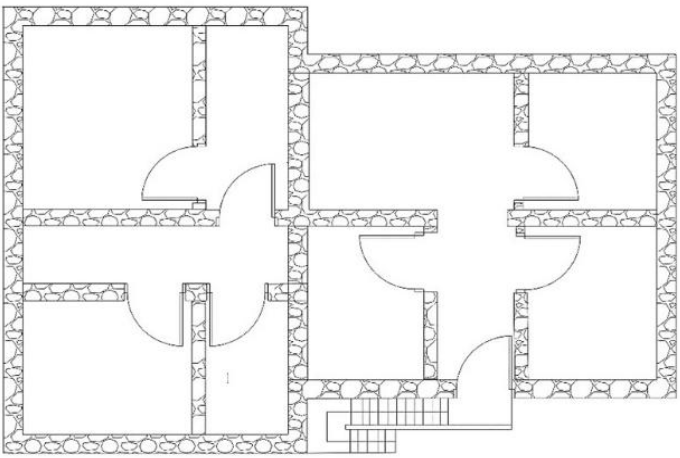

Figure 3 Sample floor plan

3. On-Site Transformed Village Houses

New houses are more spacious and open-plan. Living spaces have been integrated with functional areas such as the kitchen and bathroom. They are designed as reinforced concrete or steel. Different types have been identified in the constructed buildings. Those in need of a warehouse or barn have been offered a two-story project. The architectural and static projects of the buildings have ensured that they are more reliable (Figure 4). Energy efficiency has been supported through the facade cladding of these buildings. To ensure the longevity of the structures, high-quality materials and aesthetic principles have been emphasized.

Figure 4: Types of village houses in the transformation area

Different types of projects have been offered by the ministry and TOKI for each region. In some regions, the villagers built the constructions on their own land according to the type of project they chose, while in other regions, new villages were established (Figure5).

Figure 5 On-site transformation village house type floor plans

4. Basic Elements of On-Site Transformation Projects

For on-site transformation projects to be successful, various elements must be taken into account:

1. Earthquake Resistance: During the reconstruction process, making village houses earthquake-resistant should be a priority. Structures built with materials such as reinforced concrete, steel, wood, or stone require various reinforcement techniques to ensure earthquake safety.

2. Social Structure and Participation: In-situ transformation projects, the participation of the local population is very important. Projects shaped according to the needs of the people enhance social cohesion and strengthen the local community's commitment to the project. In this process, it is necessary to respect the villagers' traditional ways of life and to consider their wishes.

3. Infrastructure Improvements: In rural settlements, strengthening infrastructure systems after an earthquake is a major necessity. The improvement of basic infrastructure services such as

water supply, electricity, sewage, and transportation directly affects the quality of life of the people.

4. Economic Sustainability: It is important that the reconstruction projects carried out after the earthquake contribute to the local economy. The use of the local workforce, support for trade, and strengthening agricultural activities ensure the sustainability of on-site transformation projects.

5. Advantages and Challenges of on-Site Transformation

On-site transformation projects provide many environmental and social benefits in rural areas. However, these projects also bring some challenges.

Advantages

Earthquake Safety: By strengthening or rebuilding old structures, safer living spaces can be created.

Social Cohesion: On-site transformation helps preserve social bonds and contributes to the psychological healing of the people.

Environmental Sustainability: Newly constructed buildings can be built with energy-efficient materials and environmentally conscious designs can be created.

Economic Revival: On-site transformation can create new opportunities for the local economy. Job opportunities can be offered in sectors such as construction, trade, and agriculture.

Challenges

Cost: Restructuring projects initially require a significant investment.

Social Resistance: Some villagers may be attached to their old settlements and way of life, which could create resistance to the process of change.

Infrastructure Deficiencies: The existing infrastructure in rural areas may not be sufficient for reconstruction, leading to additional investment requirements.

5. Conclusions

The reconstruction of village buildings in Kahramanmaraş and its surroundings should not be limited to creating earthquake-resistant structures. Successful urban transformation should be supported by projects that meet the needs of the local population, preserve the social structure, ensure environmental sustainability, and promote economic revitalization. The success of on-site transformation projects should be based on projects and infrastructure improvements shaped by the participation of the local community. The examples examined in the study and the public's opinions reveal how important and sensitive the process of reconstruction after the earthquake is. In order to create resilient and sustainable settlements against future disasters, it is necessary to further promote and improve on-site transformation projects.

In future housing projects, it is recommended that both approaches be balanced and integrated. This can enable the preservation of cultural heritage and meet the needs of modern life. For example, in new housing projects, the use of traditional stone craftsmanship and wooden embellishments on the facades of modern reinforced concrete structures can preserve aesthetic and cultural values while ensuring modern safety standards.

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CHAPTER VI

The Impact of Primavera Risk Analysis Tool on Construction Project Duration and Cost

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Gürkan Emre GÜRCANLI[2]

1. INTRODUCTION

The construction sector plays an important role in the economic growth of developed countries like Turkey. The advancement of technology in the construction sector, coupled with the utilization of novel materials and the growing significance of vertical architecture and façade designs, has given rise to a multitude of novel risks. The control or reduction of existing and potential risks in the construction sector, coupled with the development of new risk management strategies, can prevent the potential for financial,

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temporal and qualitative losses, as well as mitigate the risk of occupational accidents**.** (Kömürlü & Güzelay, 2021) In the present research, an analysis is conducted to investigate the influence of risk management, a critical component of project management, on a tangible project throughout its lifecycle from the bidding stage to the post-construction period.

The objective of this paper is to provide a comprehensive guide that elucidates and streamlines the process of cost and schedule risk assessment in the construction industry. To achieve this, the paper employs the risk management procedure and the Primavera Risk Analysis (PRA) tool. The PRA tool offers project managers a detailed understanding of potential risks through simulations and analyses, including Monte Carlo simulations. These facilitate more accurate forecasting of probable outcomes and provide a practical framework for project stakeholders to develop and conduct schedule risk analysis, thereby reducing the probability and impact of adverse risk events in the project.

1.1 Project Management

According to the ((PMI), 2021), project management is defined as the application of knowledge, skills, tools, and techniques to project activities to meet project requirements.

Project management is a structured approach to planning, executing, and overseeing projects with the objective of achieving specific goals within defined constraints, such as time, budget, and resources. It encompasses a variety of methodologies and practices that assist organisations in effectively managing their projects, particularly in the construction industry. The implementation of project management techniques enables the identification of potential risks at an early stage of the project lifecycle. This allows project teams to develop strategies to mitigate these risks, thereby increasing the reliability of project outcomes.

1.2 Risk Management

Risk management is a project management technique that employs a systematic approach to identifying risks at the proposal stage of a project. This entails determining the impact of risks on the project by taking uncertainties into account and taking the necessary precautions in accordance with these impacts. When determining project risks, it is essential to consider various factors, including project size, complexity, location, type, and techniques used. Furthermore, risks should be evaluated in a systematic manner to ensure comprehensive and effective risk management. (BİRGÖNÜL & DİKMEN, 1996) The most fundamental reason for the increasing importance of risk management is that unanticipated increases in costs and timescales have resulted in the failure of numerous construction companies in their projects. The failure of many construction companies to complete their projects within the planned budget and time is often the result of insufficient examination of potential risks during the planning phase. This project considered the risks that could arise during budget preparation and aimed to minimise the impact of unforeseen risks through the use of the contract.

1.3 Primavera Risk Analysis

Primavera Risk Analysis (PRA) is a sophisticated tool that provides project managers with substantial benefits in the mitigation of risks and the enhancement of schedule reliability. Integration with Primavera P6 enables PRA to facilitate seamless schedule imports and facilitate the development of risk-adjusted plans through Monte Carlo simulations and scenario analysis. This results in more informed decision-making and realistic project forecasts. The capacity of PRA to model complex risks and perform sensitivity analysis, as exemplified by tornado diagrams, facilitates the identification of critical risk factors. Furthermore, the tool enhances confidence levels in project timelines, frequently achieving a 90% probability of success. Its comprehensive visualisations, including distribution graphs, facilitate the clear communication of risks and potential outcomes to stakeholders. (Duncan, 2016)

The initial stage of risk analysis is the preparation of the work schedule, which must be conducted with the utmost accuracy. The initial stage of the process entails the identification of activities and the subsequent entry of the relationships and durations associated with these activities into the programme. The work programme may be prepared using the Primavera Risk Analysis (PRA) programme or with the assistance of Primavera P6 and Microsoft Project (MSP). (Mhetre, 2017) In the subsequent phase, risks are defined. In order to determine the risks, a number of different methods are employed. These include an examination of similar projects that have been completed previously, a brainstorming session with the project team, which is comprised of experts in this field, and the experiences of the front consultant. Once the risks have been identified, the probabilities of each risk and their potential impact on time and cost are determined. Once the risk analysis has been carried out, the time and cost of the project are monitored in the best, planned and worst case scenarios. Depending on the results, a decision will be made on the management of the risk in accordance with the risk response planning.

2. LITERATURE REVIEW

Risk management in the construction industry is a very comprehensive and meticulous process. Risk management in construction projects involves identifying, assessing and mitigating potential risks to improve project performance and success. Effective risk management is critical due to the inherent vulnerabilities in construction that can lead to delays, cost overruns and quality issues.

2.1 Plan Risk Management

Risks associated with construction projects may exhibit a high degree of unpredictability. The management of risks within construction projects is acknowledged as a critical process, particularly when executed in a methodical fashion throughout the entire lifecycle of a construction endeavour. Consequently, from the initial planning phase to the final completion stage, this approach facilitates the attainment of project objectives concerning time, cost, quality, safety, and environmental sustainability. The management of risk events and overall project risks requires the formulation of strategies at two different levels. The implicit risk management plan deals with the overall project risk, which involves a thorough analysis of the project's framework, content, context and scope, while the explicit risk management plan deals with individual risk events and includes the processes for identifying, analysing, responding to and regulating individual risks. (Appiah, 2020).

2.2 Risk Identification

The initial stage of risk management entails the identification of potential risks that could impact the project. This necessitates an understanding of the various factors involved, including human resources, materials, equipment, and environmental conditions. (Rani, et al., 2024)

In this article, I will elucidate the three methods I employed in defining risk.

Personal interview: In this method, information about possible risks is gathered by experts via telephone or face-to-face meetings. This investigation is conducted with professionals or contributors involved in the project, stakeholders, and subject matter experts. The primary objective is to ascertain potential risks and to acknowledge their existence.

Checklist Analysis: Risks are identified based on the information and historical understanding that have been obtained from previous analogous projects, additional sources, and relevant data. Risks are identified on the basis of available data and historical evidence, and while this approach is useful and straightforward, it may not provide a fully comprehensive checklist. As the project progresses, newly identified risk elements that arise need to be incorporated into this checklist and improved for subsequent applications.

Brainstorming: The process entails the generation of ideas and solutions through a collective debate with all parties involved in the project, with a focus on activities related to building works and engineering constructions of various kinds. Additionally, it encompasses an examination of the risks that building and construction projects are facing, and the impact of these risks on the project's time, cost, and quality (al-Mukahal, 2020)

2.3 Qualitative Risk Analysis

Qualitative risk analysis is a methodical approach in project management used to assess the potential impacts and probabilities of identified risks. This approach is inherently subjective, relying on the expertise and insights of the project team and relevant stakeholders. Risk identification can be accomplished through various techniques, including brainstorming sessions, expert interviews, and historical data analysis. Each identified risk is evaluated in terms of its likelihood of occurrence and its potential consequences for the project. This evaluation is often carried out using tools such as risk matrices or risk probability and impact assessment frameworks. The primary advantage of qualitative risk analysis is its ability to help project managers prioritize and address risks that pose the greatest significance, thereby reducing uncertainty and improving risk management. (Akhtar, Shabbir, & Cheema, 2023)

2.4 Quantitative Risk Analysis

Quantitative risk analysis in construction involves the systematic evaluation of risk likelihood and impact through numerical data to facilitate informed decision-making. This approach enhances the identification, analysis, and management of potential risks, thereby improving project outcomes. By employing quantitative techniques, project managers are able to refine risk management strategies and optimize project performance within the construction industry. (Rajesh & Keshav, 2022). In any case, qualitative analysis establishes the core framework for operation, while quantitative analysis reveals the specific benefits of these evaluations through numerical data that underpin subsequent research efforts. Unfortunately, in practical applications, it is often the case that evaluations are conducted utilizing exclusively one of these methodologies, thus yielding an incomplete understanding of the subject matter.

2.4 Risk Response Planning

Risk response planning constitutes an essential dimension of project management, encompassing the identification of potential threats, opportunities, and obstacles, alongside the formulation of strategies aimed at mitigating or leveraging these factors. Such strategies may include the avoidance, mitigation, transfer, or acceptance of risk. The resilience and frequency of the risk management process ought to be commensurate with the complexity and significance of the project at hand (Mutula, Engairo, & Kimathi, (2023)

2.5 Monitoring and Controlling Risks

Risk management in construction projects in developing countries is essential to effectively monitoring and controlling risks. By carefully monitoring and controlling risks, project managers can ensure that projects are completed on time, within budget and to quality standards. This approach helps to achieve project objectives by reducing potential problems, optimizing the use of resources and ensuring successful project delivery. (Rashmi Jaymin-Sanchaniya, 2024.)

2.6 Monte Carlo

The utilization of Monte Carlo simulations in the analysis of project timelines has emerged as a fundamental component of quantitative methodologies for assessing project-related risks.

Monte Carlo Simulation is an invaluable tool for estimating time, start and finish times, success rate and cost in projects. This method is particularly useful for addressing complex problems that cannot be precisely solved using analytical techniques. Instead of oversimplifying a difficult problem to achieve an analytical solution, Monte Carlo Simulation provides an approximate solution through simulation. It is particularly efficacious for comprehending and quantifying the impact of project uncertainties. By employing Monte Carlo Simulation, project managers can evaluate and justify the necessity for project reserves to manage risk events throughout the project's duration. (ÇOLAK & UĞUR, 2023)

One of the fundamental principles employed in the Monte Carlo methodology is a statistical distribution. Although there exists an extensive array of various continuous distributions, the triangular distribution is applied in this particular investigation. Since the minimum, maximum and most accurate estimates for a variable can usually be readily determined, the triangular distribution, which only requires the aggregation of these data, is often favoured as one of the most appropriate distributions for use in the construction sector.

3. METHODOLOGY

3.1 Identification of Risks: Risk Identification In this investigation, a systematic literature review was initially performed, followed by the establishment and validation of the construction risk factor divided groups through personal interviews, checklist analysis, and brainstorming sessions. Document Analysis to begin with, a comprehensive examination of the pertinent literature regarding construction risk factors was undertaken, employing the keywords "construction risk in façade projects," "risk management in construction projects," "project management," "risk management plan" and "construction risk" within both domestic and international academic databases. The risks identified by the project team, drawing upon their experiences from old projects as well as their personal expertise, were evaluated within this study. Post identification of the risk factors, expertise from a façade specialist was sought, which assisted in the formulation of the conclusive version of the risk factors shaped by these expert insights.

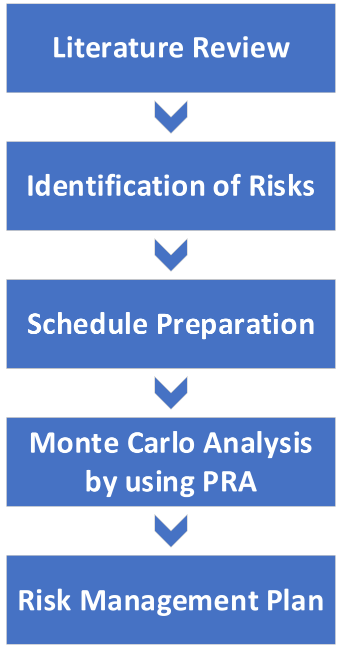


Figure 4: Methodology

3.2 Schedule Preparation

The actual schedule preparation process commences with the collation of data, including the project commencement date, the activities involved in the construction of any structural sequences, the duration of each activity, the resources required for each activity and the associated cost. The data are then entered into the software, with the relationships between the activities given in accordance with the sequence of activities collected. It is essential that the relevant parties be consulted and that the most realistic work schedule be prepared during this process.

3.3 Risk Assignment

The assessment of risks within a construction project is contingent upon the evaluation of two pivotal dimensions: risk probability or event frequency, and risk impact. The term 'probability' is used to describe the inherent chance or opportunity of a risk event coming to fruition. (Rani, et al., 2024) The assessment of risk impact on the project is defined in accordance with the potential negative consequences for the target schedule and cost. Information has been solicited from the stakeholders of the project when determining the probabilities and impacts of risks.

3.4 Monte Carlo Analysis by Using Primavera Risk Analysis Tool

As previously discussed, the project team identified the most critical risks associated with the project. Following this, the likelihood of these risks and their effects for both time and cost were allocated to the chosen activities using the triangular distribution function. Subsequently, a Monte Carlo analysis was conducted using the Primavera risk analysis tool. Following this, the impact of risk factors on project duration and cost was examined.

4. CASE STUDY

This article presents a case study of a construction façade project built in a coastal area with high winds in Muğla. The project encompasses an area of approximately 800 square meters and incorporates fixed, sliding and motorized aluminium joinery. In these joinery components, special combination glasses with large dimensions and a thickness of 38 mm are utilized. The specific glasses and joinery were selected in accordance with the particular requirements set forth by the façade consultants and the customer. The surfaces to which the joinery is affixed comprise reinforced concrete walls in certain areas and steel substructures in others. In evaluating the potential risks associated with this project, it is important to consider the unique architectural aspects, including the high aesthetic standards and the precise location sensitivity of the joinery, which is within a narrow range of plus or minus 5 mm. Additionally, environmental factors related to the project's location, work safety risks associated with the positioning of the joinery, and the potential impact of weather conditions and the timing of work permits, which may be influenced by the project's proximity to tourist areas, must be taken into account. Once the risk factors had been identified, a comprehensive work schedule was initiated. In the preparation of the work schedule, 31 activities were defined and the minimum, most probable and maximum durations of these activities were determined using the triangular distribution method. Furthermore, the resources necessary for the completion of these activities and other all cost were defined in the work schedule. The potential risks associated with this project, from the initial proposal stage to the post-construction phase, were evaluated using Monte Carlo simulation within the Primavera Risk Analyzer software. This analysis aimed to assess the impact of these risks on the project's time and cost constraints. As a result of the analysis, it was possible to evaluate the project's completion time and costs as minimum, probable and maximum.

The risks for this project were examined under four main headings, and a total of 47 risks were identified for evaluation.

Table 1: Risk Factor in Construction Façade Project

Risk Category	**Risk ID**	**Risk Factor**
Bid and Contractual Risks	**B1**	**Lack of clear detail about project**
	B2	Errors in material pricing, labour estimates, overhead calculations
	B3	submit a bid that is lower than the required amount in order to be awarded the tender.
	B4	The preparation of an offer is undertaken without the benefit of an inspection of the site.
	B5	Preparation of the offer without proper research of the regulations and specifications

	B6	A lack of comprehension of the customer's requirements
	B7	Unclear description of the work at contract
	B8	Undefined brand and certificates for materials at contact
	B9	Undefined design standards
	B10	Unrealistic construction timelines for façade installation in the contract.
	B11	Undefined the payment schedules and approvals timeline
	B12	Undefined force majeure situations (e.g., extreme weather, strikes, pandemics) in the contract.
	B13	Unprotected the budget and timeline in the event of unfrozen site conditions.
	B14	Undefined delivering the production standards

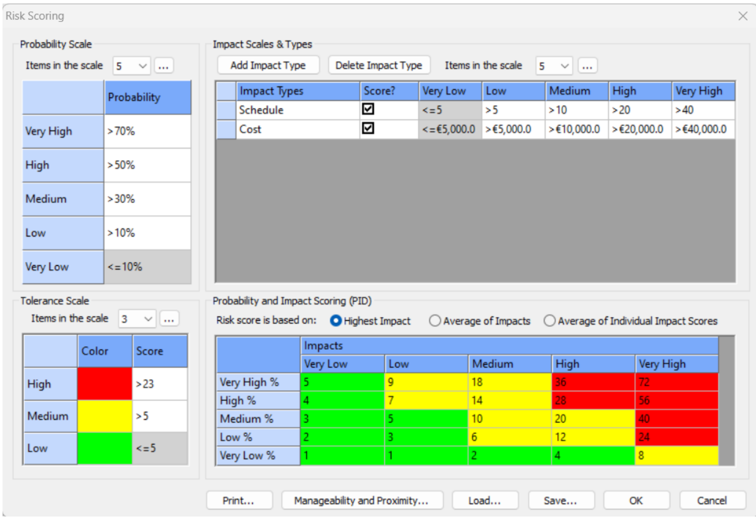
	B 15	Undefined warranty terms
	B 16	Undefined well maintenance periods
Design Risks	**D 1**	**Lack of coordination between design team, façade consultants, other specialists, and the contractor**
	D 2	**Last-minute changes or revisions requested by the client or other stakeholders**
	D 3	**Late approvals**
	D 4	Insufficient design details, miscalculations
	D 5	Using different software between stakeholders
	D 6	Inadequate consideration of specifications
	D 7	Aesthetic concerns
	D 8	Failure to consider the appropriate glass types,

		thicknesses, and coatings in relation to the aluminium framing system
	D 9	Misalignment between the client's vision, the design's practicality, and the available budget
Transportation and Custom Risks	**T 1**	**Shipping Delays**
	T 2	**Changes in Custom Regulations**
	T 3	Product standards and certifications
	T 4	Custom Inspections
	T 5	Import Licences
	T 6	Trade policies
	T 7	Political stability
	T 8	Choosing insufficient transportation company

	T 9	Documentation Issues
	T 10	Damage During Transport
	T 11	Cargo Theft
Construction Risk	**C 1**	**Weather conditions**
	C 2	**Inappropriate design for installation**
	C 3	**Inconvenient rough construction**
	C 4	**Insufficient survey support**
	C 5	Subcontractor selection
	C 6	Safety Risks
	C 7	Working at heights
	C 8	Unsuitable site conditions

	C 9	Falling objects
	C 10	Damaged materials
	C 11	Structural integrity

From the identified risks, ten risk factors were identified as being the most likely to affect the project duration and time the most. The probability of these risks, as well as their impact on duration and cost, were determined. The following table illustrates the criteria that were employed in the determination of the risk matrix. In accordance with the specified criteria, the degrees of risk were determined, leading to the generation of risk scores. Subsequently, the identified risk factors were assigned to the activities that they affect, and the subsequent analysis was conducted.



Risk Scoring

Probability Scale

Items in the scale 5

	Probability
Very High	>70%
High	>50%
Medium	>30%
Low	>10%
Very Low	<=10%

Impact Scales & Types

Add Impact Type | Delete Impact Type | Items in the scale 5

Impact Types	Score?	Very Low	Low	Medium	High	Very High
Schedule	☑	<=5	>5	>10	>20	>40
Cost	☑	<=€5,000.0	>€5,000.0	>€10,000.0	>€20,000.0	>€40,000.0

Tolerance Scale

Items in the scale 3

	Color	Score
High		>23
Medium		>5
Low		<=5

Probability and Impact Scoring (PID)

Risk score is based on: Highest Impact | Average of Impacts | Average of Individual Impact Scores

	Impacts				
	Very Low	Low	Medium	High	Very High
Very High %	5	9	18	36	72
High %	4	7	14	28	56
Medium %	3	5	10	20	40
Low %	2	3	6	12	24
Very Low %	1	1	2	4	8

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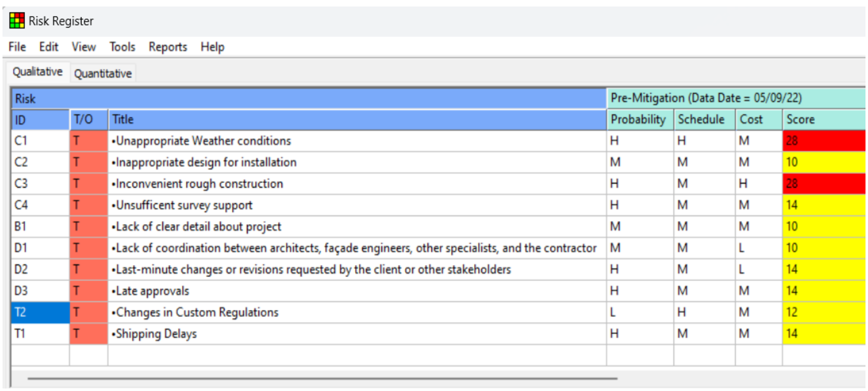
Figure 2: Risk Probability and Impact Scale



Risk Register

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Qualitative | Quantitative

Risk			Pre-Mitigation (Data Date = 05/09/22)			
ID	T/O	Title	Probability	Schedule	Cost	Score
C1	T	•Unappropriate Weather conditions	H	H	M	28
C2	T	•Inappropriate design for installation	M	M	M	10
C3	T	•Inconvenient rough construction	H	M	H	28
C4	T	•Unsufficent survey support	H	M	M	14
B1	T	•Lack of clear detail about project	M	M	M	10
D1	T	•Lack of coordination between architects, façade engineers, other specialists, and the contractor	M	M	L	10
D2	T	•Last-minute changes or revisions requested by the client or other stakeholders	H	M	L	14
D3	T	•Late approvals	H	M	M	14
T2	T	•Changes in Custom Regulations	L	H	M	12
T1	T	•Shipping Delays	H	M	M	14

Figure 3: Risk Register for Schedule Risk Model

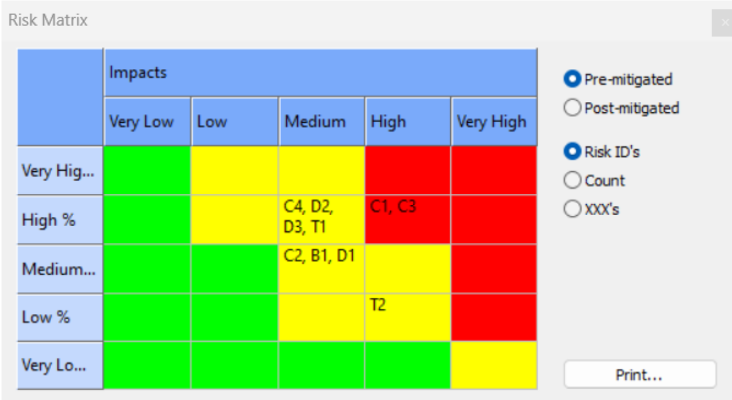

Figure 4: Risk Score Matrix

To check the scheduling, initially the schedule was calculated without considering the risks with probabilistic time and triangular distribution function.

Fig. 5 represents the completion of the whole works cumulative distribution histogram for original plan. The minimum probable project completion time was 472 days (Fig. 5). The maximum probable project completion time was 565 days. However, the most probably project completion time was 524 days at 50% probability the project completion of whole works could be finished at 09/02/24 (523 days), At 80% probability the project completion of whole works could be finished at 21/02/24 (535 days). Concerning the possible project costs, the calculated items in Fig. 5 were as follows (Fig. 6). The maximum estimated cost for the project was €741,536. The minimum estimated cost for the project was €710,594. The most likely estimated cost for the project was €727,383 (Fig. 7). The deterministic cost was €726,179.0. At 80 % probability the project completion cost of whole works could be €731,835.0.

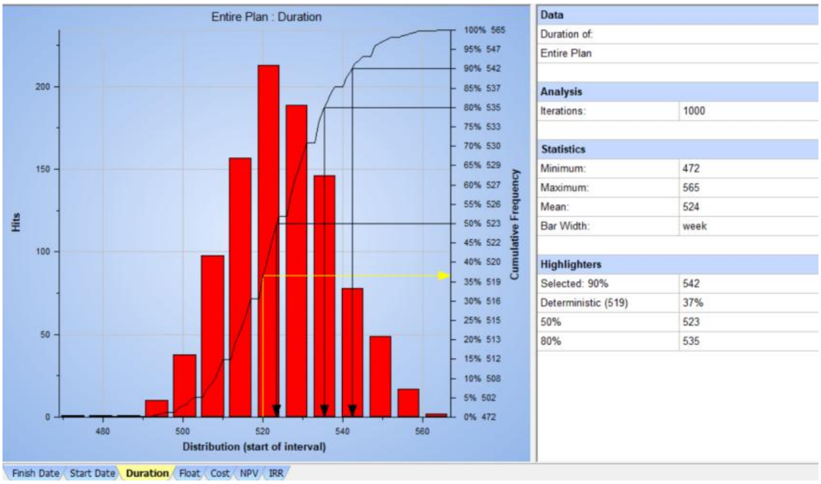

Figure 5: Original Plan Finish Duration Histogram

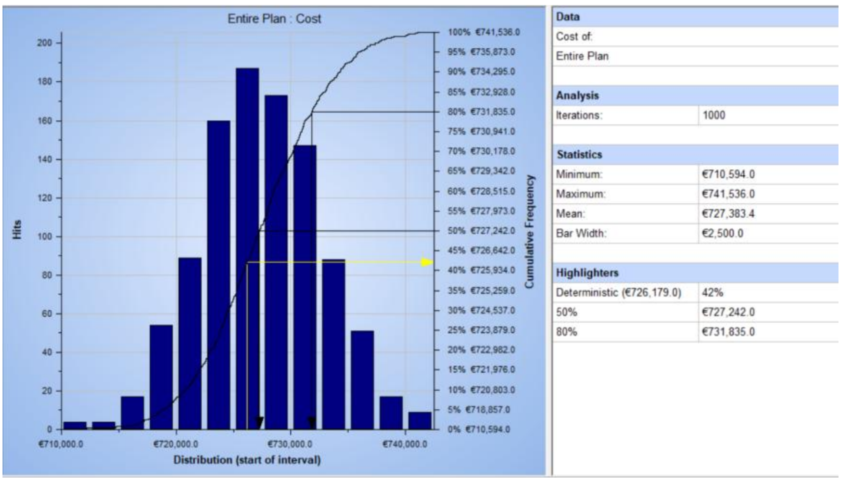

Figure 6: Original Plan Finish Cost Histogram

After selecting the preceding risks, the probability of risks and the effect of time-cost with the triangular distribution function were assigned to the selected activities, and Primavera Risk Analysis recalculated the time and cost. Fig. 7 represents the completion of the whole works cumulative distribution histogram for original plan. This histogram is the result of pre-mitigation scenario. This means that the mitigation response actions are not yet implemented. The minimum probable project completion time was 515 days (Figs. 7). The maximum probable project completion time was 675 days. However, the most probably project completion time was 584 days. At 50% probability the project completion of whole works could be finished at 08/04/24 (582 days), At 80% probability the project completion of whole works could be finished at 29/04/24 (603 days). The calculation of the project cost was verified by taking into account the risk (Fig. 8). The maximum estimated cost for the project was €931,128. The minimum estimated cost for the project was €735,199. The most likely estimated cost for the project was €829,665. in Figure 8 The deterministic cost was €726,179. At 80 % probability the project completion cost of whole works could be €857,381.

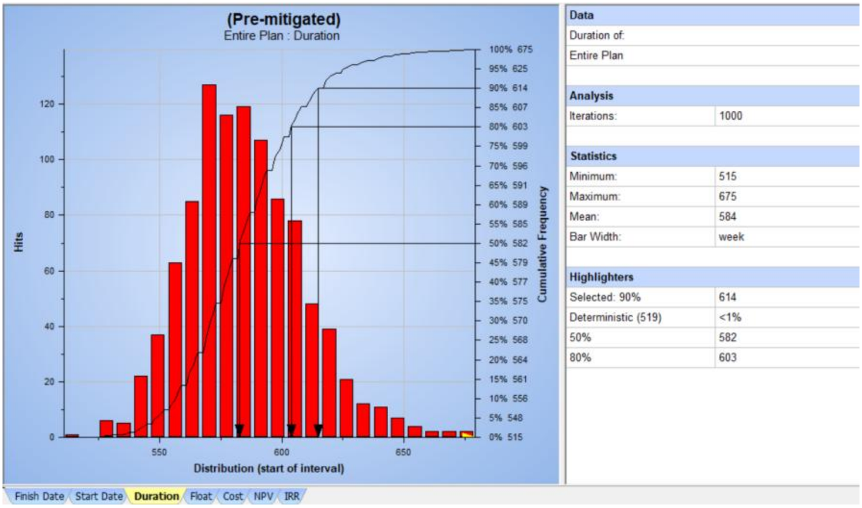

Figure 7: Pre-mitigation Finish Duration Histogram

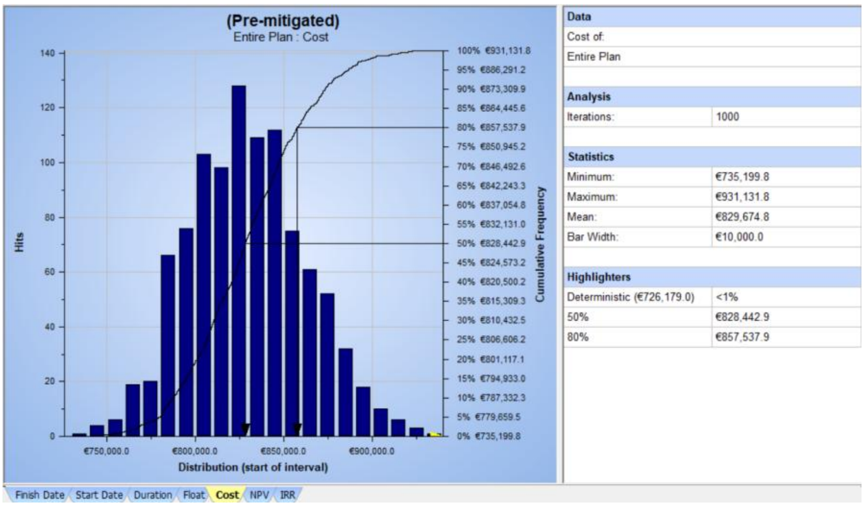

Figure 8: Pre-mitigation Finish Cost Histogram

The impact of the Primavera Risk Analysis tool on construction project duration and cost is significant. It offers a comprehensive solution to the management of uncertainties and the mitigation of risks. As posited by (Aderbag, Elmabrouk, & Sherif, 2018), the Primavera Risk Analysis tool integrates risk management processes seamlessly with project scheduling and costing, thereby enabling project managers to identify potential risks at an early stage and plan accordingly. This proactive approach serves to minimise the possibility of delays and cost overruns, while also enhancing the accuracy of project timelines and budgets. The documentation produced by Oracle elucidates how this tool's sophisticated analytics and simulation capabilities facilitate the generation of a realistic forecast of potential project outcomes, thereby enabling project teams to make well-informed decisions and adjustments at various stages throughout the project lifecycle (Oracle, n.d).

Moreover, the capacity of Primavera Risk Analysis to provide comprehensive risk reporting and analysis is vital for maintaining project control and stakeholder confidence. As the (Mhetre, Wagh, Bhujbal, Patil, & Ranaware, 2019) elucidates, this tool facilitates a structured and systematic evaluation of risks, thereby facilitating a more comprehensive visualisation of their impacts on project schedules and budgets. Furthermore, the utilisation of data from previous projects and the incorporation of real-time risk assessments enables Primavera Risk Analysis to facilitate the development of more resilient project plans. As (Primavera Consultants, n.d) posit, such comprehensive risk analysis not only streamlines project execution but also improves resource allocation and contingency planning, ultimately leading to more predictable and manageable project outcomes.

In addition to the numerous advantages offered by the Primavera risk analysis program in interpreting results, the program's accessibility was also a significant factor in its selection.

This study presents a framework for project managers to observe the impact of risk factors on project duration and cost, as a consequence of both qualitative and quantitative risk analysis, and to effectively manage these risks.

5. CONCLUSION

This case study illustrates the implementation of a risk management process in the context of façade construction projects. By employing a systematic approach to information gathering, a total of forty-seven risk events, encompassing various risk categories, were identified by the conclusion of the risk identification process. Subsequently, the project team proposed probability scales and impact scales for the prioritized risk factors. Consequently, the risk categories encompass the following: (1) Bid and Contractual Risks, (2) Design Risks, (3) Transportation and Customs Risks, and (4) Construction Risks. A quantitative analysis was conducted for the most significant risk events using Monte Carlo analysis simulation with the Primavera Risk Analyzer software. The Primavera Risk Analysis software is designed to assist project managers in the assessment of risks associated with residential building construction projects, utilizing the Probability-Impact Matrix method. The software facilitates risk identification, classification, analysis, response planning, and monitoring. It enables project teams to evaluate the impact of risks on specific tasks and activities, guiding them in determining appropriate actions to mitigate these risks effectively. In this study, the schedule was calculated with risk and without risk. Subsequently, the effect of risk factors on project time and cost was observed. Based on these findings, the project manager was able to determine the most appropriate response to the identified risks, thereby increasing the likelihood of project success.

It should be noted, however, that this research is not without limitations. These include the reliance on probabilistic assumptions, the focus on a specific case study, high aesthetic concerns, and the fact that the project is located in a tourist area. In addition, certain external factors, such as force majeure, have been excluded. It would be beneficial for future studies to expand on this work by applying PRA and Monte Carlo methods across a broader range of project types and incorporating dynamic risk factors that evolve over the project lifecycle.

In practical terms, the findings of this study emphasize the importance of integrating advanced risk management tools into everyday project management practices. Construction professionals are encouraged to adopt tools like PRA to enhance predictability and reduce uncertainty in their projects, ultimately achieving better cost and time management. As the construction industry continues to evolve with new challenges and complexities, the value of systematic risk management will only grow, making it a vital component for project success in the future.

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CHAPTER VII

Dynamic Analysis of Tapered Marine Outfall Pipe Resting on Sea Bottom Due to Wave Motion

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1. Introduction

Generally, submerged pipes located on the seabed are affected by wave and soil properties such as the depth of water to be submerged, wave height and period at this specific depth, bottom roughness along the pipe and seabed soil properties. They are also affected by pipe properties such as diameter, pipe span length, distance between the pipe and the soil, support types and additional weights, geometric and bending stiffness, and pipe material and

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structural properties. Especially considering the internal and external forces that determine the pipe size, the pipe system is designed as a pipe laid freely on the seabed, a suspended pipe and a partially or completely buried pipe. Steel pipes laid freely on the seabed are usually covered with a concrete jacket or supported intermittently using artificial concrete blocks against floating. Suspended pipes are fixed to the ground from supports. Buried pipes remain fixed in place by the ground weight on them.

In sea discharge systems, treated or raw sewage water is carried to the diffuser located in the sea by a fixed diameter pipe. Wastewater is discharged into the sea from the outlets located on these diffusers (Figure 1). As the wastewater coming out of the holes on the diffuser progresses in the pipe, its flow rate decreases and its speed decreases. This causes sedimentation in the diffuser. In order to keep the flow rate constant, diffusers are generally modeled as gradually narrowing or gradually narrowing and their hydraulic calculations are made (Öztürk, 1996).

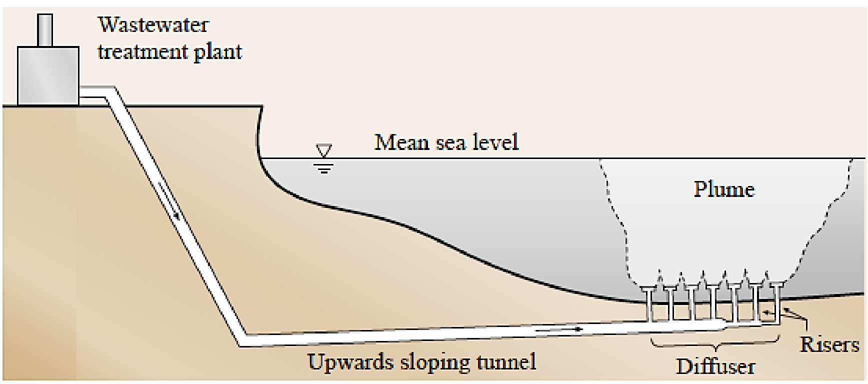

Figure 1. Schematics of a wastewater treatment plant outfall (Peter et al. 2016).

In this study, the situation of a suspended and conical sectioned pipeline in the sea discharge system line or a simply supported gradually changing diameter diffuser located at the end of the sea discharge system is taken into consideration (Fig. 2).

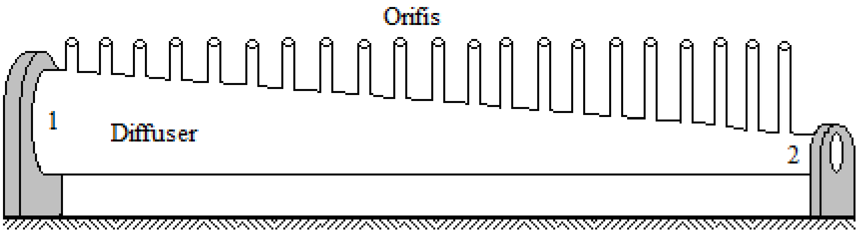

Figure 2. Gradually varied-section pipe

(Yiğit, 2010).

Fluid is transmitted through the pipe indicated in Fig.1. The control volume of pipe flow between section (1) and section (2) is selected and subjected as impermeable zone surrounding pipe walls. The the law of conservation of mass (Eq.1) and conservation of energy (Eq.2) can be written for mentioned volume. The fundamental laws of one dimensional-steady flow for constant and uniform density, and incompressible fluid can be derived by simplification of Navier-Stokes equation for conservation of mass, linear momentum and energy fitted to the streamline of flow; (Eq.1,a,b.) (Beattie et al. 1971; Yamamoto et al. 1974; Layton, 1976; Grace, 1978)

$$\frac{\partial \vec{V}}{\partial t} + (\vec{V}.\nabla)\vec{V} = -\frac{1}{\rho}\nabla p + \nu \vec{\nabla}^2 \vec{V} + \vec{f} \qquad (1a)$$

$$(\vec{V}.\nabla)\vec{V} = -\frac{1}{\rho}\nabla p, \int_{p_1}^{p_2} dp = -\rho \int_{V_1}^{V_2} V.dV \qquad (1b)$$

where ν is the constant kinematic viscosity, $\vec{f}$ is the body force per unit mass and assumed as the gravitational acceleration vector $\vec{g}$.

For steady, viscid, and incompressible fluid flow along a streamline, Navier- Stokes equation in Eq.1abc can be transformed to the Bernoulli equation in Eq.2.a,b for varied-section pipe resting on horizontal seabed.

$$\rho\left(u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}\right)=-\frac{\partial P}{\partial x}+\rho g_x+\mu\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right) \quad (2a)$$

$$\rho\left(u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}\right)=-\frac{\partial P}{\partial y}+\rho g_y+\mu\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right) \quad (2b)$$

For varied-section pipe, the basic design parameters are imposed on continuity, the Bernoulli and momentum equations as seen in Eq. 3.a,b,c.

$$V(x)=(A_1V_1)/A_{(x)} \quad (3a)$$

$$P(x)=P_1+\gamma_f\left[\frac{V_1^2}{2g}-\frac{V(x)^2}{2g}-\sum h_f-\sum h_k\right] \quad (3b)$$

$$N(x)=P_1A_1+\rho_f Q_1 V_1-P(x)A(x)+\ \rho_f Q_{(x)}V_{(x)} \quad (3c)$$

Here, the parameters are symbolized as fluid density; ρ_f, pipe area, velocity, pressure and axial reaction forces respectively in varied section; $A_{(x)}, V_{(x)}, P_{(x)} N_{(x)}$.

2. Wave Forces Acting on submerged pipe

Among the different wave theories, the suitable one, Linear wave theory, is selected for determining hydrodynamic wave forces in accordance with the water depth (d) in pipe location, design wave period (T) and height (H) at the considered depth. Linear wave theory gives water particle velocity and acceleration (u, v) components in the depth of mid-point of pipe in Eq.4.a,b.

$$u(x,t) = \frac{\pi H}{T}\frac{coshk(d+z)}{sinhkd}\cos\left(\frac{2\pi x}{L} - \frac{2\pi t}{T}\right) \quad (4.a)$$

$$v(x,t) = \frac{2\pi^2 H}{T^2}\frac{coshk(d+z)}{sinhkd}\sin\left(\frac{2\pi x}{L} - \frac{2\pi t}{T}\right) \quad (4.b)$$

Drag forces acting perpendicularly to pipe axis (F_D) (Eq.5), inertia force (F_I) (Eq.6) in horizontal plane and lifting force (F_L) (Eq.7) in vertical plane to be able to cause the periodical oscillation-induced instability of pipe can be calculated by the Morison equation depending on mentioned particle properties of wave as below.

$$F_D(x,t) = \frac{1}{2}C_D\rho_w D(x)u(x,t)^2 \quad (5)$$

$$F_I(x,t) = \frac{\pi}{4}C_I\rho_w D(x)^2 v(x,t) \quad (6)$$

$$F_L(x,t) = \frac{1}{2}C_L\rho_w D(x)u(x,t)^2 \quad (7)$$

Here, hydrodynamic coefficients on lifting, inertia and drag forces are respectively denoted as C_L, C_I, and C_D (Karal, 1977; Machemehl, 1978).

3. Stability Analysis of Submerged Pipe

Vertical forces are composed of buoyancy force F_B, pipe material and fluid weights F_P and F_F and lifting hydrodynamic force F_L. Besides, drag and inertia forces F_D, F_I and friction force F_S on soil bed under artificial supports can be described as horizontal forces (Lambe & Whitman, 1969) (Figure 3).

Soil bearing capacity F_C (Eq.8) and soil-structure friction force F_S (Eq.9) are calculated as follows;

$$F_C = \left(N_q \gamma_s z + {}^1\!/_2\, N_\gamma \gamma_s B\right) B \qquad (8)$$

$$F_s = F_V\, tan(a) + {}^1\!/_2\, \gamma_s z^2\, tan^2\left(45 + {}^{\phi}\!/_2\right) \qquad (9)$$

Here, the design parameters are taken as $Nc, Nq, N\gamma$; soil bearing capacity parameters, ϕ; internal friction angle, z; depth of structure base, γ_s; specific weight of soil, B; width of base $a = \phi/2$.

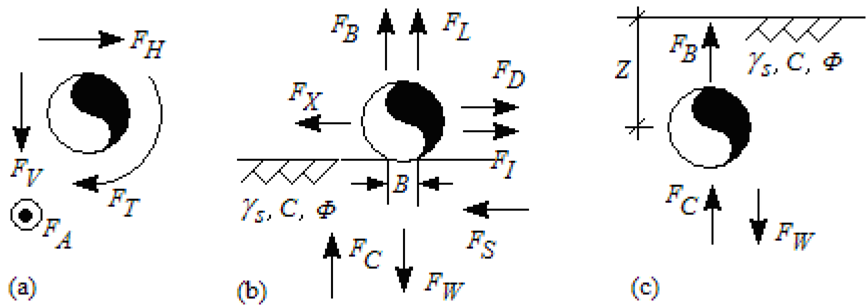

Figure 3. (a) Forces Acting on Diffuser, (b) Free Sitting Pipe, (c) Buried Pipe.

(Yiğit, 2010).

The basic stability principles are required preventing the movement of pipe and saving its balance against mentioned design loads. To provide the vertical and horizontal stability, the mentioned constraints in Eq.10 should be satisfied:

$$\sum F_V = F_P + F_F - F_B - F_L > 0 \qquad (10a)$$

$$(F_B + F_L)G_S \leq F_P + F_F \qquad (10b)$$

$$F_C \geq G_S(F_P + F_F) \qquad (10c)$$

$$\sum F_H = F_D + F_I - F_S \leq 0 \qquad (10d)$$

$$(F_D + F_I)G_S \leq F_S \qquad (10e)$$

Here, G_S is recommended as 1.5 especially in designing marine structures (Karal, 1977)

4. Governing Generalized Single-Degree of Freedom System

The basic principle is to obtain coordinates of all points on gradually varied-section pipe displaced and their displacements $v(x,t)$ (Eq.11) with varying time and position against its external moving forces; (Clough & Penzien, 1993) (Figure 4.)

$$v(x,t) = \psi(x)Z(t) \tag{11}$$

In this formula, shape function of beam-like pipe and time-varying vibration are respectively denoted as $\psi(x)$ and $Z(t)$. Therefore, the generalized single degree of freedom system (Eq.12) can be written in non-linear form as below;

$$m^*\ddot{Z}(t) + c^*\dot{Z}(t) + \bar{k}^*Z(t) = F_T^*(x,t) \tag{12}$$

Here, $Z(t)$, $\dot{Z}(t)$, $\ddot{Z}(t)$ and $F_T^*(x,t)$ are respectively indicated as displaced position of time-varying vibration, rate and acceleration of motion of pipe, and external forcing. Mass m^*,damping c^*, and rigidity $\bar{k}^*$terms given in Eq.13-15 are as;

$$m^* = \int_0^{Lp} m(x)\,\psi(x)^2 dx \tag{13}$$

$$c^* = a_1 \int_0^{Lp} EI(x)\,\psi''(x)^2 dx \tag{14}$$

$$k^* = \int_0^{Lp} EI(x)\,\psi''(x)^2 dx \tag{15a}$$

$$k_G{}^* = \int_0^{Lp} N(x)\,\psi'(x)^2 dx \tag{15b}$$

$$\bar{k}^* = k^* - k_G^* \tag{15c}$$

Here, the mentioned parameters are mass of pipe and transmitting fluid $[m(x)]$ and inertia moment functions of varied section of pipe $I(x)$ and generalized elastic and geometric rigidities k^* and k_G^*, length of pipe (Lp), modulus of elasticity (E), axial compressive force (N) and damping coefficient (a_1). External force acting on the system can be adopted to the governing equation of motion as follows (Clough & Penzien, 1993; Celep & Kumbasar, 2001). For crossflow motion in Eq. 16, for in-line motion in Eq. 17;

$$F_T^*(x,t) = F_L(x,t) - F_P(x) - F_F(x)]\psi(x)dx \tag{16}$$

$$F_T^*(x,t) = \int_o^{L_P} [(F_D(x) + F_I(x,t)]\psi(x)dx \tag{17}$$

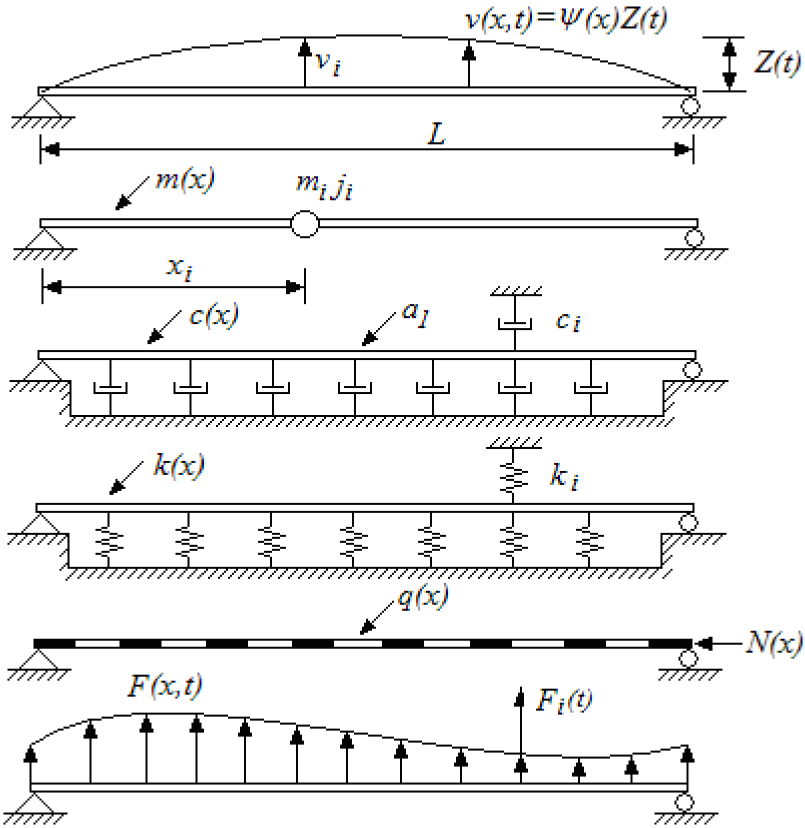

Figure 4. Generalized Single Degree of Freedom System (G-SDoF)

$$F_T^*(x,t) = F_L(x,t) - F_P(x) - F_F(x)]\psi(x)dx \tag{16}$$

$$F_T^*(x,t) = \int_o^{L_P}[(F_D(x) + F_I(x,t)]\psi(x)dx \tag{17}$$

5. Numerical Analysis

In this study, design pipe parameters which is located at 25 m depth are sorted pipe span length 9.00 m, inlet pipe diameter 1.00 m, outlet diameter of pipe 0.60 m in addition to simply supported beam-like pipe which is manufactured as varying-section tapered (conical) steel material (Figure 5). Besides, pipe, sea and fluid properties are additionally presented as in Table 1-4 (Ölmez et al., 2011; Yiğit et al., 2017; Gücüyen et al., 2020; Yiğit & Gökkuş, 2024).

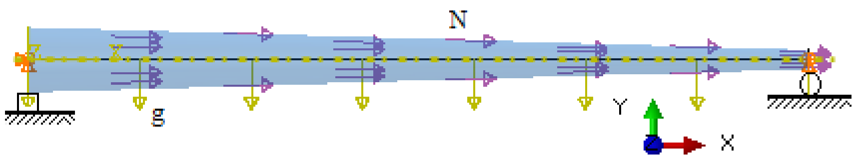

Figure 5. Marine outfall system end point: tapered diffuser pipe.

Table 1. Material section and dimension of pipe.

L(m)	D1 (m)	D2 (m)	s (m)	E (kN/m2)
9.0	1.00	0.60	0.01	2.1x108

Table 2. Pipe material and fluid properties.

γ_P (kN/m3)	ρ_P (t/m3)	γ_F (kN/m3)	ρ_F (t/m3)
78.34	7.98	10.00	1.019

Table 3. Wave Parameters.

γ_W (kN/m3)	ρ_W (t/m3)	H (m)	T (sn)	d (m)
10.09	1.029	2,5	8	25

Table 4. Soil Properties.

γ_S(kN/m3)	ϕ	N_c	N_q	N_γ	B m	C (kN/cm2)
20	30o	37.1	22.4	19.1	0.15	20

By analysing these parameters, the pipe motion is generated by Generalized Single-Degree of Freedom System (G-SDOF).

5.1. Vertical and Horizontal Stability of Pipe

Hydrodynamic lifting, inertia and drag force coefficients of suspended pipe can be assumed as $C_L \approx 0$\{for e/D>1/4\} while taking $C_I = 2.2$ and $C_D = 1$ in all cases (Yamamoto et al., 1974). Hydrodynamic lifting force, buoyancy force, weight per unit length of pipe volume filled with fluid, weight per unit length of pipe and weight of artificial concrete pipe jacket per unit length are respectively calculated as $0, 7.925, 7.392, 2.436$ and $2.059\ kN/m$.

By applying vertical balance Eq.35, the required stability condition is satisfied to Eq. 10.a,b,c. For total beam-like pipe length $L_P = 9.0\ m$, total vertical net force and bearing capacity of soil under each support will be $35.668\ kN$ and $208.000\ kN$. Safety factor for vertical stability of pipe should satisfy to Eq.10.b,c so that the bearing capacity of soil is calculated as $G_S = 2.4$.

For horizontal stability of pipe mentioned in Eq.10.d,e, hydrodynamic drag force $0.073\ kN/m$, hydrodynamic inertia force 0.335 kN/m, total horizontal wave force $0.450\ kN/m$ and soil friction force $32.124\ kN/m$ for partially buried (about 0.20m-depth) artificial supports in cohesionless soil can be calculated as expressed. This is accepted as highly reliable.

5.2. Determination of Longitudinal Reaction Forces

Internal fluid pressure is about $60.12\ kN/m^2$. By computer-aided hydraulic design of pipe system, fluid velocities, internal pressures and reaction forces depending on x distance are computed as shown in Table 5.

Table 5. Longitudinal parameters of gradually varied section pipe

x	D (m)	V (m/sn)	P (kN/m2)	N (kN)
0	1,00	1.3257	61.2	0
1	0.9556	1.4547	61.1577	4.1081
2	0.9111	1.6034	61.0865	3.9288
3	0.8667	1.7762	60.9848	3.7452
4	0.8222	1.9784	60.8501	3.5576
5	0.7778	2.2173	60.679	3.3664
6	0.7333	2.5022	60.4665	3.1719
7	0.6889	2.8458	60.2048	2.9743
8	0.6444	3.2653	59.8821	2.7741
9	0.60	3.7849	59.4794	2.5711
			Total (N)	30.197

Total longitudinal reaction force due to impuls-momentum effect of tapered pipe is about $30\ kN$.

5.3. Dynamical Analysis of the Time History in SAP2000 Software

First of all, the mentioned hydrodynamic forces are loaded to the beam-like and gradually varied pipe system. Time History Analysis (THA) which take place in the scope of the Structural Analysis Program (SAP2000, 2011) software is performed for solving the static system under these loads. During this analysis, the weight of pipe includes steel and coating material weights. Time history solutions are useful for both linear and nonlinear structural response and sinusoidal and non-sinusoidal loading. In order get the exact solution of the THA and increase accuracy, successive time intervals preferred should be shortened.

Coordinates of laterally displaced system with respect to the unloaded pipe axis coordinates before loading are analysed numerically by using Curve Fitting Technics (CFT). From this study, the function of lateral displacement, on the other hand, the elastic curve function of system is found for initial condition of the THA.

5.4. Dynamic Analysis with G-SDoF System

5.4.1. For crossflow motion

By assuming based on the new function obtained from normalization of this function, the shape function is derived in terms of solution of the G-SDoF system. As a function of x variable, shape function $\psi(x) = [0; 1]$ can be defined as in Eq.18;

$$\psi(x) = 1{,}093\, sin(0{,}367x - 0{,}034) + 0{,}142\, sin(0{,}588x + 2{,}876) \tag{18}$$

Therefore, the generalized mass and rigidity of system [Eq.19] can be numerically found from Eq.13 and Eq.14;

$$m^* = 3{,}036, \quad \overline{k^*} = 26331{,}477 \tag{19}$$

Total vertical force mentioned in Eq.17 with changing x coordinate and time (in kN) can be written as in Eq.20;

$$\mathrm{F}_T^* = 8{,}899 + 69\cos(-0.785\mathrm{t})^2 \tag{20}$$

Governing equation of motion as in Eq.12 can be derived in Eq.21 by taking as $Z(t) = 0, \ddot{Z}(t) = 0$ for initial condition at $t = 0$ as;

$$3{,}03.\ddot{Z}(t) + 26331{,}47.Z(t) = -8{,}89 \tag{21}$$

From this equation, displacements changing with time, Z(t) can be drawn as presented in Figure.6. in which the maximum displacement is found as approximately $Z = 3{,}379x10^{-4}\ m$. (Figure 4).

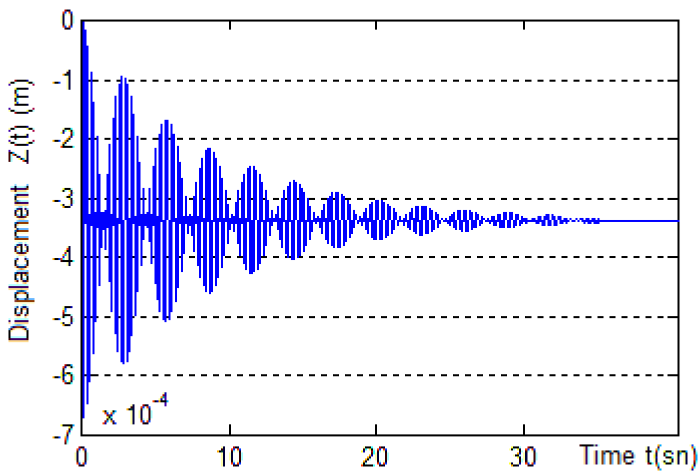

Figure 6. Displacement in time-varying cross-flow motion.

5.4.2. For in-line motion

At the equatorial level of pipe, the wave speed (m/s) and acceleration (m^2/s) in functional forms as seen in Eq.4a and Eq.4b are the following;

$$U = 0{,}377\, cos(-0{,}785t) \quad (22a)$$

$$\dot{U} = 0{,}296\, sin(-0{,}785t) \quad (22b)$$

In order to obtaine the generalized drag and inertia forcings (kN) mentioned in Eq.6 and Eq.7, they are found by utilising from Eq.23 and Eq.24;

$$F_D^*(t) = 0{,}335\, cos(-0{,}785t)^2 \quad (23)$$

$$F_I^*(t) = -1{,}234\, sin(-0{,}785t) \quad (24)$$

Governing equation of motion (Eq.12) is finally developed as below.

$$3{,}036.\ddot{Z}(t) + 26331{,}477.Z(t) = 0{,}335\, cos(-0{,}785t)^2 - 1{,}234\, sin(-0{,}785t) \quad (25)$$

The governing equation of pipe motion can be solved both by analytically from the MATLAB and SAP2000 software (Matlab, 2024; SAP2000, 2024). These three solutions have compared each other as indicated in Table 6-7.

Almost 10s later, the results dealt with in Figure 7, have entirely fitted each other. In accordance with those of G-SDOF system even though either results from analytical solutions or the SAP 2000 outputs can be taken as a basis, the CFT in Figure 8 produces the time-varying displacement function defined as in Eq.26.

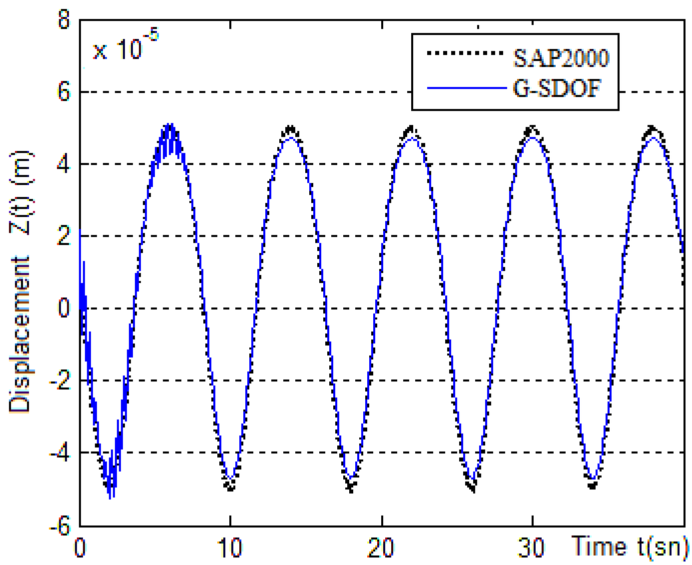

Figure 7. Comparison of results from G-SOF and SAP2000 for in-line motion

If compared oscillations on crossflow motion with those on in-line motion, it is seen that the prevailing oscillations is generated from in-line motion.

For prevailing condition on the beam-like pipe system, time-varying displacement can be described in the form as seen in Eq.26;

$$Z(t) = 4{,}685 \times 10^{-5}. sin(0{,}785t - 3{,}1) \qquad (26)$$

In order to get both coordinate and time-varying displacement function $v(x, t)$ in Eq.28, this function has to multiply with shape function mentioned.

$$v(x,t) = (1{,}093\, sin(0{,}367x - 0{,}034) + 0{,}142\, sin(0{,}588x + 2{,}876)) \times (4{,}685 \times 10^{-5}. sin(0{,}785t - 3{,}1)\, \psi) \qquad (27)$$

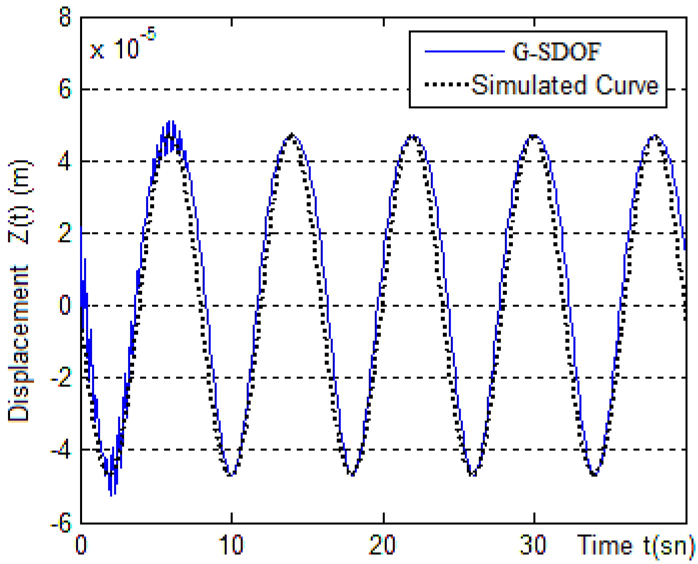

Figure 8. The time-varying displacement function for in-line motion because of the prevailing condition

Finally, this derived function represents the distance-time varying in-line oscillations of system which are greater than those of crossflow oscillations.

6. Conclusion

The G-SDoF results generated from horizontal and vertical forcing terms and those of the THA technics based on the SAP 2000 software are fittingly derived. Both figures obtained from itself results can be compared to regard these suitability as in Table 6 and Table 7.

Table 6. Maximum displacement in vertical plane (crossflow motion) (for x=L/2)

	G-SDoF	SAP2000	Difference (%)
Displacement (m)	3,379x10-4	3,732x10-4	9,65

Table 7. Maximum displacement in horizontal plane (in-line motion) (for x=L/2)

	G-SDoF	SAP2000	Difference (%)
Displacement (m)	4,68(×10-5	5,07(×10-5	9,94

The differences between both of two displacements are close each other. The basic cause of this accuracy can be said that the elastic curve of simply supported beam-like pipe system as the assumption of shape function is chosen at the beginning of calculation.

Besides, the SAP 2000 software gives the accurate displacement value changing with time at the certain point on the pipe axis. It does not produce the required function. Intervals between points to be considered should be shortened in order to get more reliable results. Time-varying displacement function of each point can be derived from SAP2000 but this cannot be defined as both distance-varying and time varying function. To get displacement function with both x and t variables, the relation between their functions of all points considered should be established.

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CHAPTER VIII

Arch Theory in the Historical Process Of Structural Analysis

Abdullah ÇETİN [1]
Şahnaz TİĞREK[2]

1. Introduction

Remains of Old Stone Age (Palaeolithic) cultures are found in various parts of Anatolia, especially in Southern Anatolia. Archeologists report that around 7000 BC, the new stone (Neolithic) age, along with agriculture and animal husbandry activities started (Kuban, 1978). The richness of the mines, which are the natural building blocks of Anatolian geography, has also enabled the development of stone art.

Migrations have taken place to discover new homelands and new geographies since the creation of human beings and throughout their life cycle in the world, and wars have become inevitable as a

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result of migrations. They built palaces, city walls, castles, and bridges out of masonry structures to secure their permanence in their homelands. They have found and developed new methods with the help of environmental conditions to gain superiority over their opponents in wars (Çulpan, 2002).

In Greek architecture, which is also frequently encountered in Anatolia, there is a design based on geometry, measure, and proportion, and perfection has been achieved by repeating these three criteria over the years. Another important building heritage has been left by the Romans. They created a style that greatly improved the construction technique of the arched, vaulted, and domed buildings, added importance to the interior spaces of the buildings, and built the monumental building order on these construction techniques (Kuban, 1978).

The arch application dates back several thousand years. There are some suggestions in the literature about the origin of the arches ((Heinrich, 1983);(Kurrer, 2002)) According to one suggestion, arches were formed as a result of the fall of the false vaults, and according to the other, it was assumed that the arches were formed as a result of the disintegration of the lower parts of the supporting stone elements or stone beams into individual pieces as shown in Figure 1.1 (Özkaya, 2019).

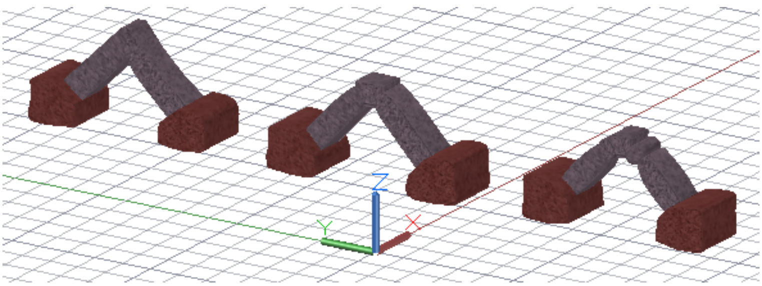

Figure 1.1. Schematic representation of the formation of arch from stone rocks

The medieval masons developed their skills through practice and passed on their experience to the next generation through apprenticeship rather than written documents (Lourenço, 1998). As a result of the study of the available structures, some knowledge was gained about the mastery of the assembly and use of the arches.

The most important of these is the necessity of a keystone when making the arches, a total number of stones forming an arch would be an odd number, and, the arches were formed by masoning the stones with the help of molds. Today, these molds can be made from two different materials, wood or steel, with the development of technology Further information on molds can be found somewhere else (Heyman, 1995).

The five-minute theorem for masonry: If the wood center of an arch is removed after the masonry work has been completed and the work remains stable (not demolished) for five minutes, then it will stand for 500 years. The boundaries of the wall contain the thrust line and are satisfied by the safe theorem. It can be deduced that the upper limit of 500 years is the decay life of the material. Thus the stability of the masonry structure will be ensured primarily by the shape of the structure and not by the strength of the component material mortar (Heyman, 1995).

The statement of (Heyman, 1995) can be updated with the rule of ten minutes and 1000 years of stability. Because there are masonry structures in our country and the world, they are older than 1000 years.

The masonry arch cannot support its weight until all the arches are in place. In the phase of application, the weight of the arch is carried by the centering with wood or steel material. When the centering is removed, the arch will begin to work structurally. The first few stones which are can sustain themselves. As the arch moves inwards, a newly placed stone will be in danger of slipping and a need for centering will arise. Any stone in the completed arch loop can be considered a keystone that ensures the stability of the whole. However, the center stone is called the keystone. Because it is the last stone that enables the arch structure to be formed (Heyman, 1995).

Using mathematics in the design of the structures emerged with the Renaissance (Heyman, 2006).

2. Material and Method

This study aims to better understand the arch theory by drawing solid models by transforming two-dimensional drawings into three-dimensional drawings, important studies of arch theory, which started in the seventeenth century and shaped until today. Mathematical formulas and equations are not included in the present study.

It is assumed that structural analysis began with Galileo, who first introduced the concept of stress. Although Villard and Vitruvius had architectural studies on arch theory, there is no information about the calculations related to arch theory in the written sources that have survived in terms of structural analysis (Heyman, 2006). On the other hand, it was Robert Hook who first introduced the arch theory. The main researchers who contributed to the development of the arch system by making important theoretical studies on arches; The findings of Robert Hooke (1675), Blondel (1683), David Gregory (1697), Gautier (1717), La Hire (1695), Couplet (1729) were examined in detail by Heyman (1982, 1995, 1998, 2006). A summary of these studies will be given in this section. Hooke; In 1675, analyzed mathematically the chain curve (catenary) and gave the solution in the form of an anagram: “A flexible chain will look like a solid arch when suspended and inverted”. The math of the statics of a stretched hanging rope is the same as that of an inverted arch. Hooke was concerned with the actual mathematical and mechanical form of all kinds of arches for construction. Thus a possible inverted chain represents one of the infinite number of ways in which the arch located within the boundaries of the semicircular arch in Figure 2.1 can carry its weight (Heyman, 2006).

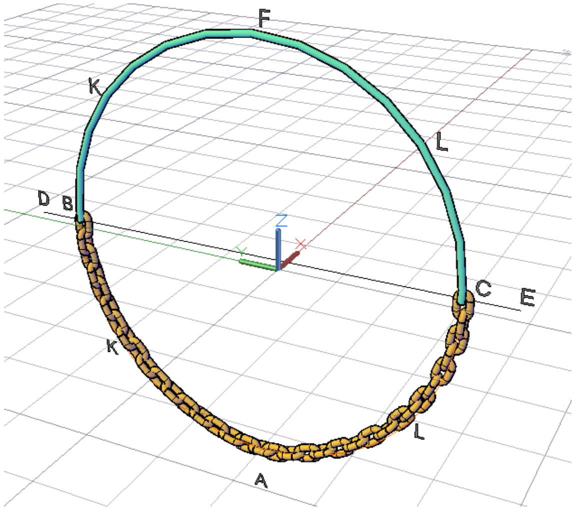

Figure 2.1. Robert Hooke's hanging chain three-dimensional solid model

In 1683, Blondel stated (in Figure 2.2) that shallow arches provide more thrust, require thicker abutments, and a round arch can push abutments less than a pointed arch (see Figure 2.2) (Heyman, 1995). Davut ÇETİN, who is a masonry foreman living today and has no theoretical knowledge about the mechanics of masonry structures, states that the pointed arch can carry more load.

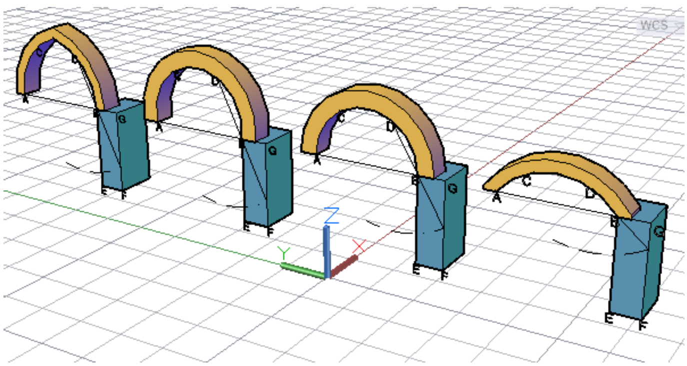

Figure 2.2. Three-dimensional solid model of Blondel's rule

According to Gregory (1697), the horizontal component of an arch's bearing force has the same value as the horizontal pull exerted by the equivalent hanging chain. The reverse hanging chain is called the thrust line of an arch and represents how pressure forces are transmitted from the wall to the supports. Once this thrust line is placed for a particular arch, simple static equations allow the magnitude of the arch's thrust force to be calculated and the design can be completed (Heyman, 2006).

The first known work on bridge piers was published by Gautier in 1717. According to Gautier; The five challenges for bridge arches are; the thickness of the abutment, the dimensions of the inner buttresses as a ratio to the arch span, the thickness of the arch ring, the shape of the arches, and the dimensions of the retaining walls to hold the soil (Heyman, 2006). La Hire in 1695 when examining the statics of a semi-circular arch made of joined stones assumed that the connections between the stones were frictionless (Figure 2.3.). He presented the problem of finding the weight of the semi-circular geometric arch to maintain static balance and concluded that for an arch with straight arches, the thrust line must be perpendicular to the joints. He then showed that by working backwards, a polygon of forces could be formed and finally the weights of the stones making up the arch could be found. Later in 1712, La Hire abandoned the assumption of smoothness and considered the friction to be a magnitude that would not allow it to slip. As a result, the direction of the thrust line inside the arch is no longer perpendicular to the joints, and there is no simple starting point for the static analysis. The purpose of the analysis was to determine the value of the arch pressure so that the abutments could be designed, and he stated that the arch would break somewhere between the spring and the keystone (Heyman, 1998).

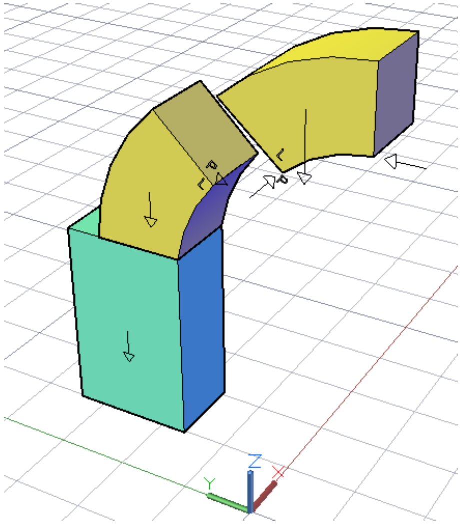

Figure 2.3. Static of La Hire's three-dimensional solid model arch

Couplet published two important papers on arch published in 1729 and 1730. The first study was a replication of La Hire's frictionless state analysis. In the second study, he made three basic assumptions about the behavior of the wall; the wall has no tensile strength, the wall has infinite compressive strength, and the shear failure cannot occur (Couplet, 1729).

Couplet's first theorem states that if the beam remains within the thickness of the arch and does not intersect the inner curve, the arch will not collapse, regardless of the magnitude of the load applied to the arch. Figure 2.4 shows the supporting forces and straight thrust lines (ANC and APC). ANB and APC can be generated directly from abutments B and C following straight thrust lines. For the arch to collapse, the angle BAC needs to be widened and this can only be achieved by spreading the abutments (Couplet, 1729).

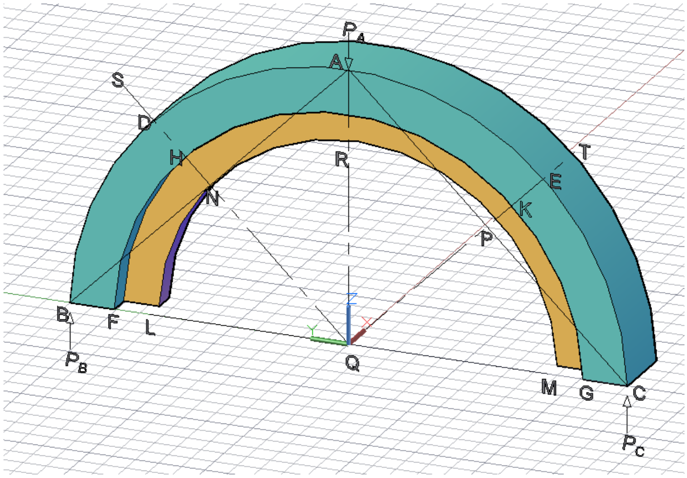

Figure 2.4. Solid model of Couplet's arch

The next problem tackled by Couplet was to find the minimum thickness of a semicircular arch bearing only its weight. Figure 2.5 is a solid model of the Couplet's three-dimensional arch. An arch that is divided by hinges DI, AH, and EK shows the hinges. Hinges IK and EK are placed at 45° on the arch forehead. The relation between the thickness, t, and the radius was proposed by (Couplet, 1729).

$$t/r = 0.101. \qquad (2.1)$$

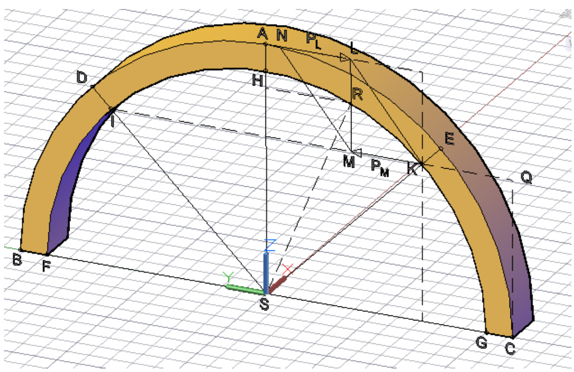

Figure 2.5. Solid model of Couplet's three-dimensional arch

Castigliano formulated energy principles to find solutions in elastic structures in 1879. He stated that if a structure is hyperstatic and there are an infinite number of solutions to support a given applied load, then the true elastic state of the structure is the state where the stored strain energy will be minimal. Among other practical examples, he applied elastic energy theorems to a masonry bridge, by taking advantage of the elastic properties of stone and mortar by allowing cracking if the thrust line is outside the middle third of the cross-section (Heyman, 2006).

In 1948, Pippard observed that the abutments of a stone arch would spread very slightly, thus he analyzed with hinges on the abutments. Figure 2.6 shows a two-hinged parabola with loading on the centerline of the arch (Pippard, 1948).

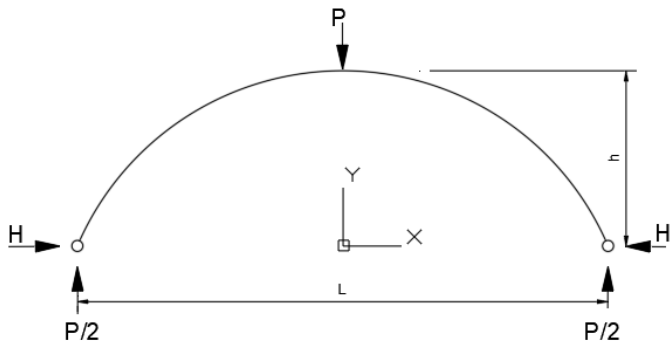

Figure 2.6. Two-hinged arch system

The theory of structures deals with the mechanics of slightly deformable bodies. If a structure is to be of practical use, its displacement must be very small. The structure as a whole and its components are not completely rigid, and therefore very small deformations generate large internal forces. Internal forces must satisfy equilibrium equations that combine their values with those of the external loads. However, for a hyperstatic structure, the equilibrium equations have an infinite number of solutions (Truesdell & Truesdell, 1984).

Structural theory is based on static equations and external forces must balance internal forces. The structural theory deals with the principles and methods by which the direct stress, shear and bending moment, and deflection can be calculated at any section of each structure member (Wang & Eckel, 1957).

The geometric boundary conditions must satisfy the equilibrium condition along with deformations and the material responds to the applied load.

In Figure 2.14 (Villarceau, 1853) three-dimensional solid model of the yield surface formed between the stone fragments in the arch with a finite crushing strength is shown. Yield surface *BAC* is at the boundary of the *ADFEA* curve formed by the two parabolic arcs. The yield surface between the two pieces of stone forms 10% of the ADFEA parabola and the yield in this region is the safety limit of the arch. However, only a small part of this real yield surface is valid for practical masonry structures. A typical value of stress used in the design of large bridges in the nineteenth century is 10 percent of the crushing strength (Villarceau, 1853).

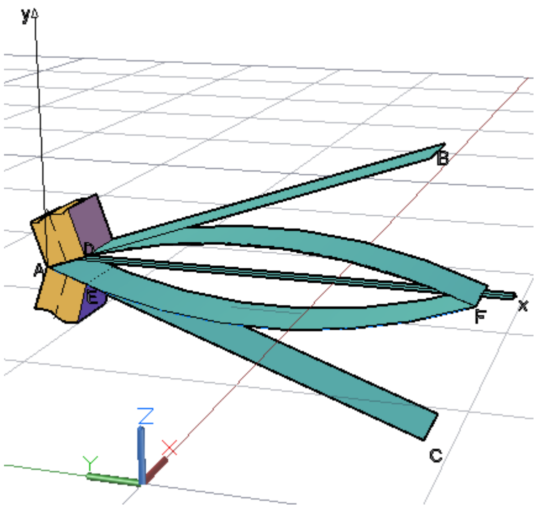

Figure 2.7. Solid model of yield surface on the wall

The safe plasticity theorem states that, whether or not straight or slightly curved boundaries are used, if all stress resultant yield surfaces are *DAE* triangles, then the structure will be safe and will not collapse. In terms of arch construction, if it can be shown that any of the infinite number of shear lines that balance the applied loads are within the arch profile, then this is proof that the arch cannot collapse under these loads (Heyman, 1995).

Analysis of arches using plastic theory shows that their safety does not depend on the conditions in the abutments or whether the arches were originally constructed to exactly match them (Heyman, 1982).

(Heyman, 1982) tried to construct a stone arch theory based on geometry. According to this theory, an arch made of any reasonable stone, with or without mortar, can comfortably accept the applied loads. Therefore, there is no problem of crushing the material in the arches. What is the best arch shape to carry the given loads? How thick should the arch loop be? What loads can a particular medieval bridge carry? He developed the limit analysis method based on plastic theory introduced by (Pippard, 1948) to find answers to questions such as:

Limit analysis

The first step is to define the elements of a masonry bridge. In Figure 2.8 arch parts (segment), keystone, fill, the wall at the top of the arch (spandrel), the lower surface of the arch (intrados), the upper surface of the arch (extrados) and the arch opening are given according to (Heyman, 1982).

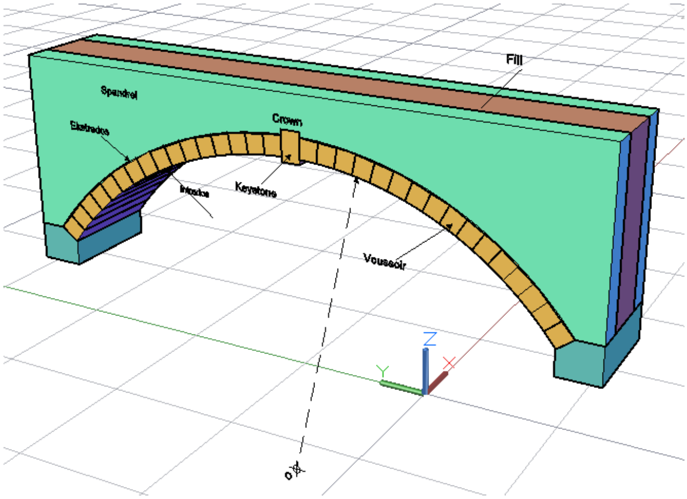

Figure 2.8. Elements of a three-dimensional masonry bridge

With limit analysis, analysis can be made by knowing the unit volume weight of the material and the geometry of the structure, based on the plastic theory.

In a small bridge, the fill may consist of rubble or gravel built to the desired height to support the road surface. The embankment is protected by angular walls built on the arch rings on both sides of the bridge. Fill structurally cannot move, but in practice, a load applied to the road surface can spread across the embankment before it is applied to the upper surface of the arch. The idea that an arch must have a minimum thickness to contain a thrust line for certain loads is key to establishing a safety factor for practical design (Heyman, 1982).

The safety theorem (Heyman, 1995) states that the arch can carry a point load of any intensity at the apex. The counterpart to this theorem is that a hinge is at the top and no hinge arrangement would result in a four-bar chain mechanism of the type shown in Figure 2.9. The four-bar chain is the basic mechanism for an arch to collapse.

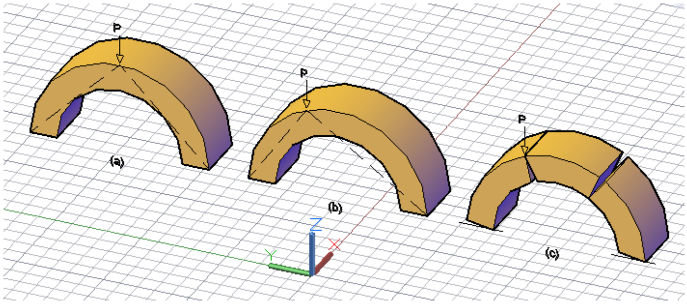

Figure 2.9. Three-dimensional solid model of arch forming four hinges

The plastic structure theory underlying the limit analysis takes into account three assumptions;

a) Balance equations must be strictly met and internal forces in the arch must balance external loads and self-weight.

b) Any statement regarding the behavior of the material should be compatible with three simplifying assumptions: The material is infinitely strong in compression, cannot take up tension and slip will not occur.

By the principles of plastic theorems, these assumptions are on the safe side.

c) The movements of the abutment are accepted as unknown.

The position of the thrust line depends on the pivot thrust. Thus, its value will change from the minimum when the arch expands to the maximum when the arch shrinks as shown in Figure 2.10.

Each position of the thrust line corresponds to a different abutment thrust value. If the arch is spread, the thrust decreases to a minimum value and if the arch is forced to decrease in span, the thrust increases to a maximum. If the abutment moves a little closer together instead of slightly apart as shown in Figure 2.10 (a) then once again hinges will form. Some crushing of the stone may occur, or blistering and splitting may occur. Such spalling can sometimes be seen under the crown (keystone) of a bridge, as shown in Figure 2.10 (b) (Heyman, 1982).

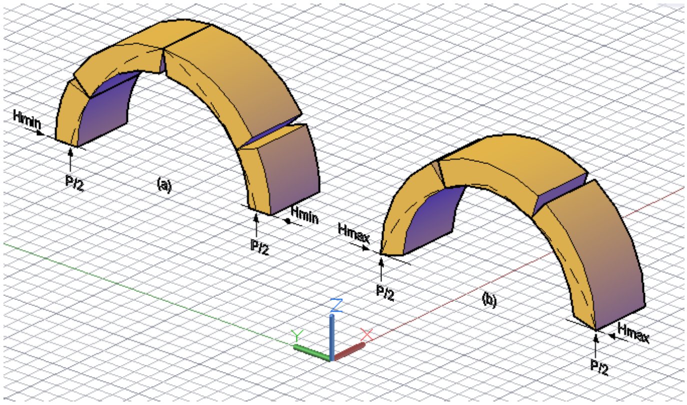

Figure 2.10. Three-dimensional solid model of maximum and minimum thrust forces in a full round arch

It is possible to translate the basic ideas from the theory of plasticity in idealized material into terms applicable to masonry. First, one can think of how the simple arch could collapse. In Figure 2.18(a), the semicircular arch moves with its weight and an additional point load *P*. Thus, Hooke's hanging chain will deform as shown in Figure 2.11 (b). As *P* increases, the inverted chain (push line) will fit less comfortably on the arch, and at a given *P* value, the push line can only be brought under control. This limiting step is shown in Figure 2.11 (c). The thrust line reaches its upper and lower surface from four locations. A hinge will form in each of these locations and the four hinges will transform the fixed arch structure into a collapse mechanism. According to the theory of limit analysis, the arch must be in equilibrium, and the mathematics of equilibrium is represented by Hooke's inverted chain, the thrust line. The material requirement was that the line of the compression had to be inside the wall. If such a position can be found for the thrust line, the structure will be stable and it is suggested that the arches will never collapse under load. Therefore, it can be assumed that the arches are strong.

Arches only fail when the loads they carry increase to the point of crushing or one of the stones slip from the structure (Heyman, 1995).

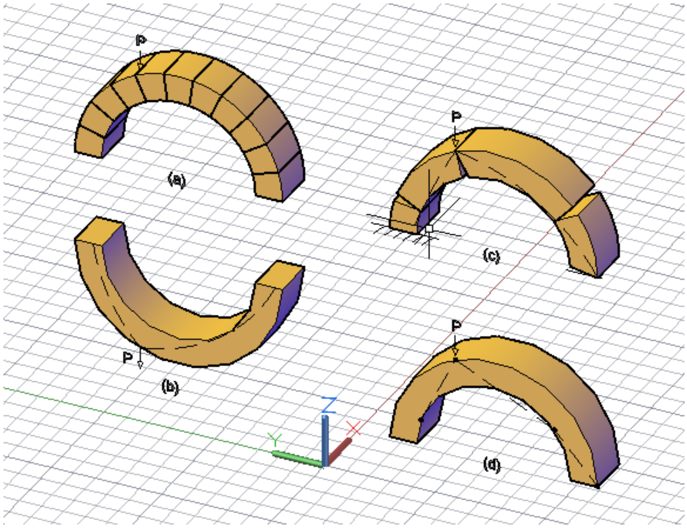

Figure 2.11. Three-dimensional solid model of a circular arch collapsing under a point load

(Frunzio et al., 2001) performed a three-dimensional (3D) analysis of a masonry arch bridge using the nonlinear finite element method. The stress levels in the arch provided information about the fracture mechanism and suggested that there was no failure of the material and that the arch may have a six-hinge failure mechanism similar to the limit analysis.

(Orduña & Lourenço, 2003) presented a new mathematical model for an arch made of rigid blocks (kinematic theorem).

(De Luca et al., 2004) proposed a simplified method for assessing the seismic capacity of masonry arches. The algorithm they developed combines linear finite element analysis and strategy limit analysis including Heyman's hypotheses.

(Gilbert, 2007) made a state-of-the-art study on the application of limit analysis to masonry structures and stated that it has been an effective method for the safety of structures.

Hand-based limit analysis techniques have left their place for computer-based methods. Various specialized analysis programs have been developed over the years based on limit analysis theorems. Currently available tools include ArchieM (Obvis 2007) and ring2.0 (LimitState 2007b). In Figure 2.12, thrust lines and hinges formed as a result of limit analysis in a masonry wall and masonry arch under point loading are shown by drawing a three-dimensional solid model. Limit analysis is a conceptually simple and robust model for analyzing the eventual collapse of masonry arch bridges. To date, most researchers have focused on establishing the stability of the wall arches, buttresses, and buttresses that make up an arch bridge. Studies in this area are generally carried out using push line (static method) or discrete rigid block type (kinematic method) analysis procedures.

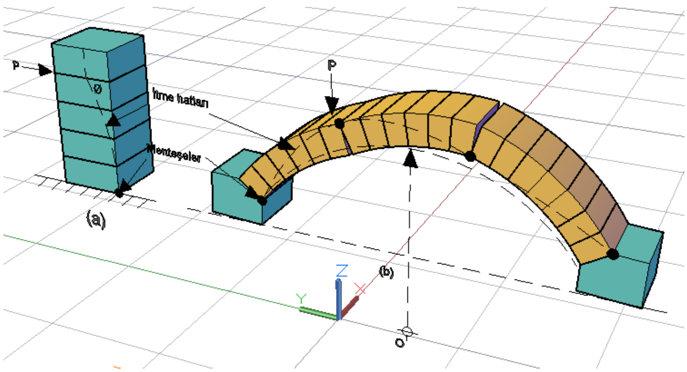

Figure 2.12. (a) The masonry wall and (b) The thrust line and hinge formation in the three-dimensional solid model of the masonry arch

(Brencich et al., 2007) showed that masonry arches often carry much higher boundary loads than expected. In arches; analyses were carried out using the linear elastic analysis method for the estimation of the stress state and the most commonly used limit analysis methods for the estimation of the boundary load.

After comparing three methods, Lourenço made several recommendations: Choose simplicity over complexity, use an analysis tool that can be validated and evaluated by the user, model parts of the structure rather than the whole structure, do not use fully structured three-dimensional models unless necessary, and do not perform linear elastic analysis on historic buildings (Lourenço, 2021).

In the study of (Çetin, 2023) the nine-century historical stone arch bridge was compared by making structural analysis with two different methods. These analyses are limit analysis and nonlinear finite element macro analysis methods. According to the findings, it was seen that limit analysis gave a quick and practical result in estimating the ultimate collapse strength of the structure compared to nonlinear finite element analysis.

3. Conclusion

In the present study, a brief review of the development stage of the arch theory has been introduced with the aid of solid model drawings.

After Hooke, in the seventeenth century, some more study was done work on arch theory in the sense of structural analysis. In the eighteenth and nineteenth centuries, arch theory focused on experimental proof as well as theoretical studies.

Theoretical work in the first half of the twentieth century was devoted to both improving and simplifying approximate methods. With the development of computers and numerical modeling techniques, these theoretical studies came to an end and the mathematical equations of elastic structural analysis were formulated in compact terms. In the nineteenth century, equations in the form of large arrays were processed by computers.

In the twentieth century, the limit analysis method, of Pipard was developed by Heyman. In the twenty-first century, the hand-based calculation procedure of the limit analysis method was further developed, by transferring it to the computer environment.

According to (Pippard, 1948) when there are three hinges, the arch collapses. According to (Heyman, 1982), when there are four hinges, the arch collapses. According to, (Gilbert, 2007) when six hinges are formed, the arch collapses.

As a result, the limit analysis method, which emerged depending on the arch theory, is the best analysis method when there is a lack of data, and it can be accepted as an indispensable fast application method in the estimation of the collapse load of the arch system.

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CHAPTER IX

Evaluation of the Critical Moment Capacity of Non-symmetric Z-Sections: Analytical and Numerical Analyses

Mehmet Fethi ERTENLİ[1]

1. Introduction

The flexural behavior of unsymmetrical beams is more complex than that of symmetrical beams, especially due to asymmetrical section properties and loading conditions. In such beams, additional effects arise due to the non-symmetry of the section axes.

If the section of a beam is not symmetrical about the bending plane or the load does not pass through the center of the beam in the bending plane, it causes unsymmetrical bending. This above effect may not only induce bending but buckling of the beam. When a load does not pass through the shear center, it also produces a torsion effect on the beam In addition to the combined bending. The flexural behavior of such beams relates directly to stability associated with lateral torsional buckling (LTB) as well as local buckling. Behavior

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of unsymmetrical steel sections in load application differs considerably from symmetrical sections because of geometric irregularities. This chapter reviews the existing literature on the flexural behavior of these unsymmetrical steel sections, their critical design parameters, and experimental/theoretical methods of analysis.

2. Non-Symmetric Bending Behavior

Bending behavior of unsymmetrical steel sections is one of the engineering problems that have attracted innumerable analysis efforts employing the finite element method (FEM). The FEM has actually stood in good stead as a very effective means to solve difficult detailed structural problems and handle information regarding the mechanical behavior of steel sections. Basically, flexural behavior of unsymmetrical profiles is affected by local buckling, general buckling, and material characteristics, etc. (Demir, 2020; Raswitaningrum et al., 2023; Pham & Hancock, 2010). Another consideration in using finite element method analysis is that many parameters will influence the flexural strength of profiles. The stresses in the section will vary based on absorbed geometry characters, eccentric loading conditions, and elastic/plastic behavior of the material; these parameters directly affect the flexural behavior of the beam (Fieber et al., 2020; Chen et al., 2019).

Stress distribution of beams under bending can be analyzed in two categories as elastic and inelastic. Elastic stress distribution occurs under loads within the elastic limits of the material and shows a linear distribution in the cross section. Inelastic stress distribution is a stress distribution that occurs when the material reaches the yield stress with the increase in the load on the element and plastic load sharing is activated, which increases the load carrying capacity of the section.

Biaxial bending is when a member is subjected to simultaneous bending along both principal axes (u-axis and v axis) In this case, the bending moments (M_u ve M_v) occurring simultaneously along both axes cause a complex stress distribution on the element. Such effects are often encountered in bridges under wind loading, or in unsymmetrical columns, or in bending elements with cross-sections without symmetry axis. As an engineering approach, the total stress in the section is determined as the superposition of the bending moment induced stresses along both axes as follows.

$$\sigma = \pm \frac{M_u v}{I_u} \pm \frac{M_v u}{I_v} \tag{1}$$

Here; I_u and I_v are the moments of inertia with respect to the principal axes of the section; u and v are the distances of the point where the stress is calculated to the neutral axis according to the section geometry.

Biaxial bending becomes more pronounced, especially in sections with no axis of symmetry (e.g. Z profiles). It has been reported in the literature that the absence of axes of symmetry leads to a more complex distribution of stresses and deformations in the section. Unsymmetrical steel sections, such as I-beams with unequal flanges, are more prone to lateral torsional buckling (LTB) than symmetrical sections. In particular, the buckling behavior of unsymmetrical sections plays a critical role in the design of such sections. Several studies have investigated different design approaches and material combinations to improve the flexural strength of unsymmetrical sections (Abambres et al., 2014; Huang & Young, 2013). In this context, the effect of lateral bracing is one of the main considerations when analyzing the flexural behavior of steel beams. Studies in the literature have generally focused on the behavior of beams constructed using double symmetry axis sections. A comparative study conducted by Zewudie (Zewudie, 2022) indicates that laterally supported beams exhibit significantly higher strength than their fully unsupported counterparts. This is more true with unsymmetrical profiles, which have a higher tendency for LTB with absent or poorly spaced lateral bracing. The study emphasizes that the design of lateral bracing must be adequately considered to ensure the stability and strength of such beams. The nonlinear finite element analzy sis presented by Zewudie highlighted yet another interaction between local buckling and LTB in cellular steel beams. Their findings indicated that if local buckling of the of the body and LTB fail modes are occurring concurrently, then surely the design flexural strength of those beams could be reduced, and such failure modes should be considered more cautiously in design practice (Zewudie, 2022). In another study led by Ertenli et al. (2021), lateral torsional buckling was evidenced by determining the limit lengths of elastic and inelastic lateral torsional buckling in steel beams with double symmetry axes under the influence of web distortion using the finite element method. As the slenderness of the section body increases, local buckling dominates the body section. In this case, combined local buckling of the body works along with LTB and the element collapses under loads smaller than the expected critical load under LTB.

In the investigation of the flexural behavior of unsymmetrical steel profiles, it is important to compare the results obtained by FEM with experimental data. These comparisons increase the accuracy and reliability of FEM and allow for better results in the design process (Moćko & Brodecki, 2018; Katwal et al., 2018). For example, studies on the behavior of cold-formed steel members under bending and shear show that the results obtained with FEM are consistent with experimental findings (Pham & Hancock, 2010; Huang & Young, 2013). The finite element method stands out as an important tool in such analyses and increases the reliability of steel structures (Shames & Dym, 2017; Wald, 2023).

Galishnikova & Gebre (2019) emphasize the importance of steel structure design specifications in establishing design curves that account for the unique characteristics of non-symmetrical sections. Their comparative study emphasizes that lateral torsional buckling is the primary limit state to be controlled for steel beams and underlines the necessity of accurate critical stress calculations.

The critical stress for LTB is also affected by the presence of voids in the body section of steel beams. Karmazínová et al. (2013) conducted an analysis of the case of lateral torsional buckling in thin-walled cold-formed steel beams and showed that such changes significantly affect the effective bending and torsional stiffness of the beams. This finding is especially true for non-symmetrical sections, since the presence of body voids further increases the risk of buckling in the member.

Experimental studies, such as those conducted by Zhang et al. (Zhang et al., 2017), further elucidate the behavior of unsymmetrical beams under point loads. Their findings on the lateral torsional buckling strength of welded single symmetric beams contribute valuable empirical data that can be used to refine theoretical models and improve design practices.

3. Finite Element Method

The swift advancement of computer technologies within civil engineering has resulted in transformative alterations in design, analysis, and application methodologies. The implementation of these technologies enables scholars and engineers to examine intricate structural systems more efficiently and with greater precision, concurrently yielding substantial improvements in safety and cost efficiency. In contemporary research, a significant number of studies are conducted utilizing computer assistance. Upon reviewing various studies within the realm of civil engineering literature (Ertenli & Köseoğlu, 2024; Şal et. al. 2024; Battal Şal & Cubuk, 2022; Olabi et al., 2021), it becomes evident that the incorporation of computer technologies into civil engineering has unveiled new opportunities in both theoretical and practical aspects of the field.

The Finite Element Method, extensively employed in addressing civil engineering challenges, represents a robust numerical analysis technique designed to address the constraints faced in the analytical resolution of intricate engineering issues. Finite Element Analysis (FEA) is employed across various domains, including structural analysis, thermal behavior, and dynamic load effects. It offers thorough solutions by decomposing complex engineering challenges into smaller, more manageable components. This has emerged as an essential instrument for evaluating structural integrity and functionality with remarkable precision, particularly within the field of civil engineering. In the present investigation, the critical buckling moment and critical stress of a Z-section profile lacking an axis of symmetry were determined utilizing the Abaqus finite element software.

When creating a finite element model, it is of great importance to define the geometry correctly, to determine the material properties to reflect the actual behavior and to select the appropriate element type for the type of problem. The mesh structure should be adjusted to optimize solution accuracy and computation time, and finer mesh structures should be used, especially in regions where high stresses or deformations are expected. It is critical for the reliability of the analysis results that the boundary conditions and loadings accurately represent the real situation, and it is important to validate the model with analytical solutions or literature results. Appropriate selection of solution parameters and methods and evaluation of the results in accordance with engineering principles ensure that the finite element analysis is valid and reliable.

The yield strength of the steel material used in the study was defined as 235 MPa, modulus of elasticity as 200 GPa and Poisson's ratio as 0.3. In order to observe the plastic behavior, the ultimate strength of the material was defined as 360 MPa and the unit deformation at rupture was defined as 0.19. A 3 m long beam was modeled in Abaqus proram using ZNP100 cross-section beam geometry. Simple support conditions were defined at the ends of the beam and it was loaded vertically from the shear center of the section at the mid-span level. The section geometry generated in the program is given in Figure 1.

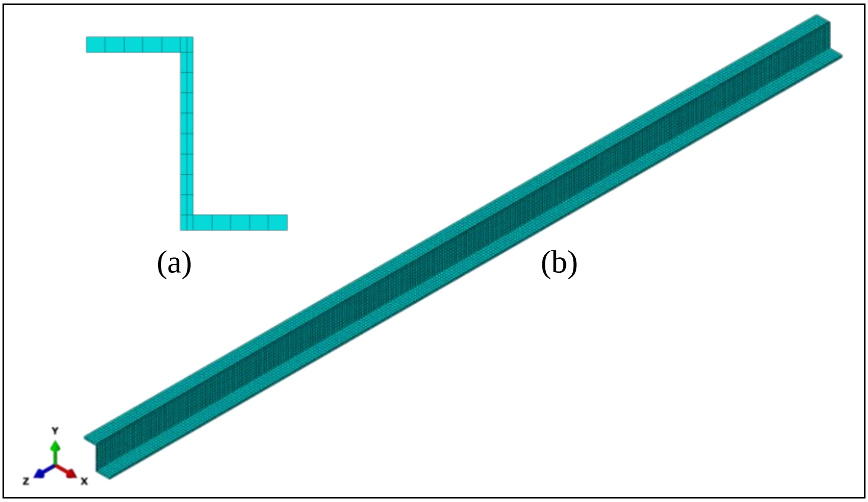


Figure 1: Z-section abaqus model; (a) sectional view, (b) 3D view

After defining the element and material model, the analysis steps were created. In order to determine the critical load of the beam, buckling analysis, which does not consider material nonlinearity, was initially performed. Mode 1 of the eigenvalues obtained from the buckling analysis corresponds to low critical load values and this mode shape shows how the beam is susceptible to buckling. The 1st mode shape and the result obtained from the buckling analysis are shown in Figure 2. As a result of the analysis, it was determined that the beam would buckle under a load of 31917 N. According to the obtained buckling analysis result, the critical moment (Mcr) of the beam was calculated as 23.94 kN-m. The buckling analysis of Abaqus is performed as a linear stability analysis and does not consider material or geometric nonlinearities. This analysis method assumes that the system remains within elastic limits and exhibits linear behavior.

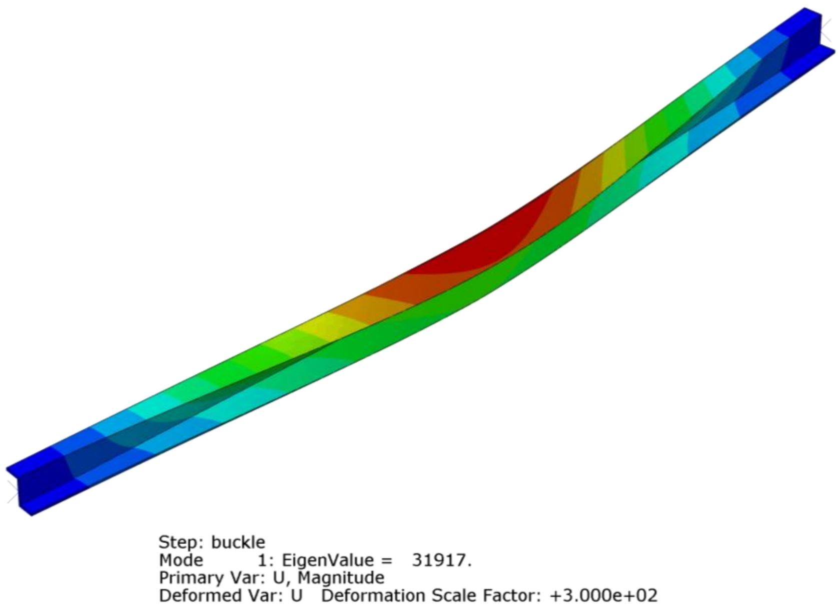


Figure 2: Z section buckling analysis result

In order to take into account the material and geometric nonlinearity, a nonlinear static analysis was also performed and the load/lateral displacement graph of the beam was obtained as a result of the analysis (Figure 3).

As a result of the nonlinear static analysis, the maximum load value of the beam was determined as 13481 N. In this context, the critical buckling moment (Mcr) of the beam was calculated as 10.11 kN-m.

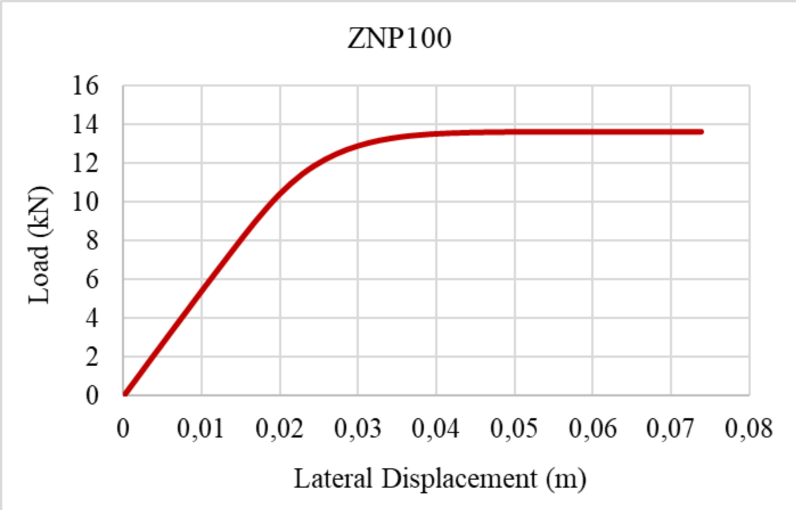
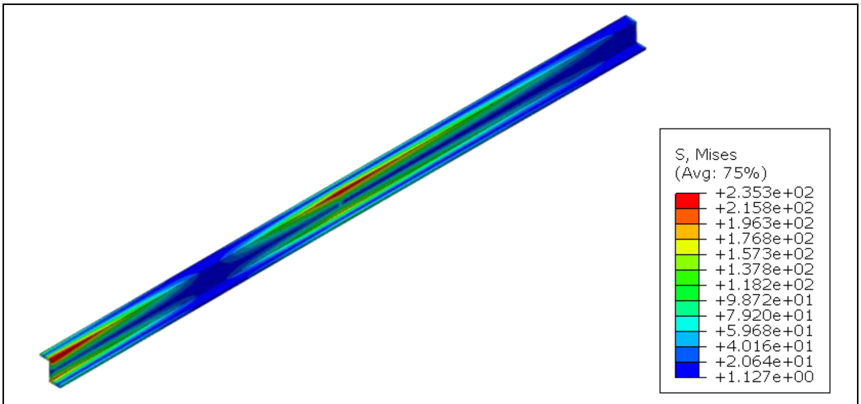


Figure 3: Load/lateral displacement graph of the beam after nonlinear static analysis

When yielding in the material behavior and nonlinear effects in the element geometry are included in the analysis process, a significant reduction in the lateral buckling capacity of the beam is observed. According to the results of the nonlinear analysis, the critical moment value decreased by 57.77% compared to the case where these effects were ignored. This shows that nonlinear effects, especially the plastic behavior of the material, geometric large deformations and loss of stability, have a significant effect on the element performance. Consideration of these effects in the design process will lead to more realistic and safer designs. The results of the analysis emphasize the effect of nonlinear effects on the lateral torsional buckling capacity of the beam and show once again the importance of the differences between the idealized analysis based on the regulations and the actual engineering conditions. The stress distribution in the beam at the end of the nonlinear static analysis is shown in Figure 4.


S, Mises
(Avg: 75%)
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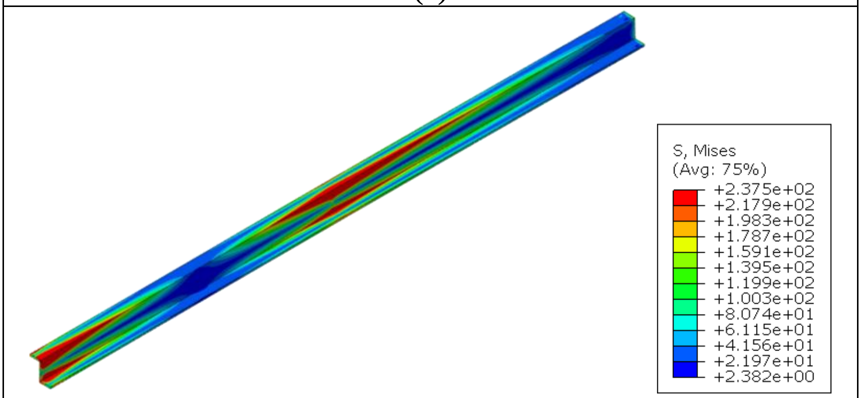
(a)


S, Mises
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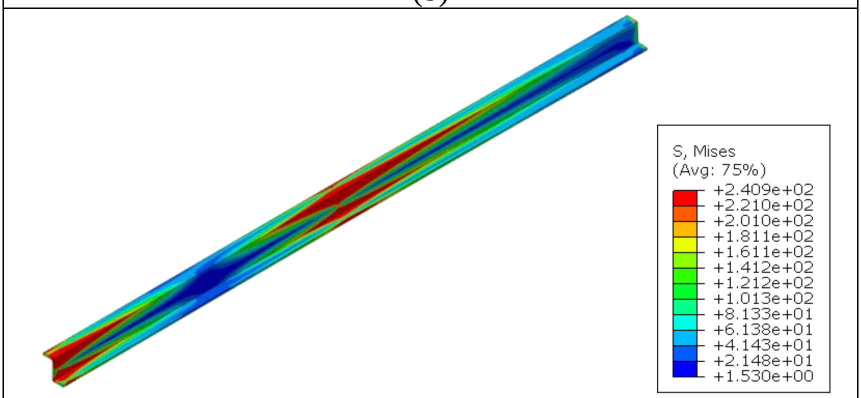
(b)


S, Mises
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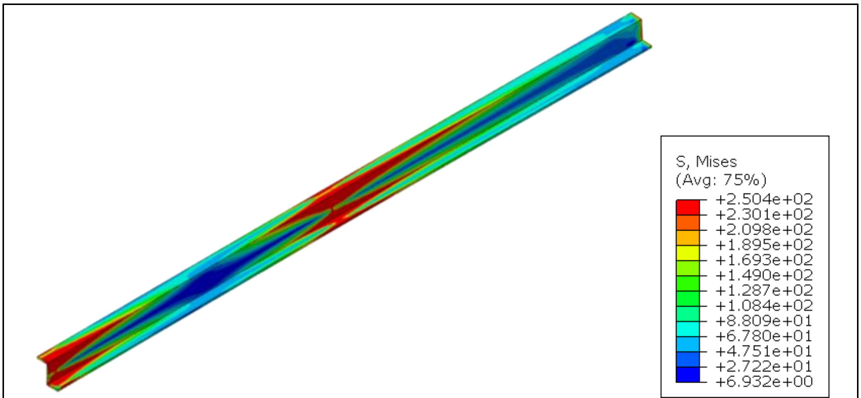
+1.530e+00

(c)



(d)

Figure 4: Stress distribution as a result of nonlinear static analysis; (a) first yield, (b) web section reaches yield stress, (c) web section completely plasticizes, (d) final step of analysis

4. Solution to the Code AISC-360-22

There are no specific provisions in AISC 360-22 to evaluate the lateral torsional buckling and local buckling of asymmetric sections (e.g. Z-section members). In this context, engineers are advised to design such sections using appropriate literature and advanced analysis methods.

Regarding the lateral buckling behavior of Z-shaped sections, the relevant User Note of AISC 360-22 recommends that the critical stress to be used in the calculation of the buckling capacity of these sections should be taken as half of the critical stress of channel (U) sections with the same flange and body properties. This approach seems to be aimed at a more conservative consideration of the torsion and stability problems caused by the geometrical asymmetry of the Z sections and the distance between the shear center and the loading axis.

In this context, the critical stress of the U-section profile with the same flange and body section length and thickness values will be calculated according to equation (2).

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0{,}078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2} \quad (2)$$

The Lateral torsional buckling (LTB) modification factor will be calculated from equation (3) for a simply supported beam under concentrated force as shown in Figure 5.

$$C_b = \frac{12.5\, M_{max}}{2.5\, M_{max} + 3M_A + 4\, M_B + 3\, M_C} \quad (3)$$

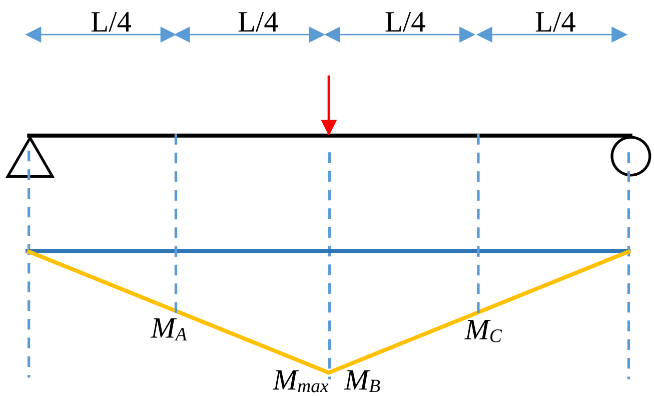


Figure 5: Moment diagram of a simply supported beam under singular load at mid-span

The moment diagram of the beam shown in Figure 4 exhibits a linear distribution. This shows that the loading and boundary conditions on the beam are regularly reflected in the moment variation. According to AISC 360-22, the moment values M_A, M_B and M_C used in the calculation of the moment modification factor (C_b) were determined in accordance with this linear moment diagram. Due to the linearity in the moment diagram, the moments M_A and M_C are equal to half of the maximum moment value (M_{max}). The moment M_B is equal to M_{max}. As a result of the calculations made by considering the effect of the linear relationship on the moment modification coefficient, C_b=1,315.

This value indicates that the regularity of the moment distribution and the constant moment gradient on the beam increase the capacity of the beam to resist lateral buckling. In this context, this definition of C_b is in full compliance with the theoretical framework provided by the code and reliably models the effects due to the nature of the moment diagram.

According to the user note in the regulation regarding Equation 2, where the critical stress will be calculated, the expression in the square root can be taken as 1 in order to stay on the safe side. In this context, Equation 5 can be used to calculate the critical stress of the equivalent channel section.

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \tag{5}$$

AISC360-22 regulation can be consulted for the definition of the notations included here, in addition, the equations recommended in the regulation for some parameters to be used in the calculations are as follows.

Effective radius of inertia "r_{ts}";

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \tag{6}$$

The strength parameters of the U cross-section considered for the calculation of the critical stress were calculated using a section designer software as follows:

Torsional constant (J) = 25023.5 mm^4

Warping constant (C_w) = 3.36884E+09 mm^6

Elastic section modulus taken about the x-axis (S_x) =43756 mm^3

When the regulation equations and the section strength parameters obtained from the section designer program are substituted in Equation 5, the critical buckling stress of the equivalent channel section's is obtained as follows.

$$F_{cr} = \frac{1.315\pi^2 200000}{\left(\frac{3000}{29.353}\right)^2}$$

F_{cr} =248.495 MPa was calculated as. This result belongs to the critical buckling stress equivalent channel (U) section, i.e. $F_{cr,U}$=248.495 MPa.

For the ZNP100 profile considered within the scope of the study, the critical stress of the U profile with equivalent flange and body cross-sectional properties was calculated as $F_{cr,U}$ =248.496 MPa using the method recommended in the regulation, and by reducing it by 50% as recommended by the regulation, the critical stress for the ZNP100 profile is obtained as $(F_{cr,Z})$ 124.248 MPa.

Based on the critical stress, when we consider the critical moment according to Equation 7;

$$M_{cr} = S_x F_{cr} = 43756 \cdot 123.248 \cdot 10^{-6} = 5.437 \tag{7}$$

The critical moment for the ZNP100 section is calculated as 5.437 kN-m.

5. Evaluation of the Results

In this study, the lateral torsional buckling (LTB) behavior of a beam with ZNP100 section is analyzed by three different methods and the results are compared. These methods are linear buckle analysis by finite element method, nonlinear static analysis and analytical calculation approach recommended by AISC 360-22. The results obtained reveal the differences between the methods as well as the important criteria to be considered in the design of lateral buckling behavior of Z-section profiles.

5.1. Buckle analysis (linear stability analysis):

The linear buckle analysis performed by the finite element method obtained the critical moment as 23.94 kN-m. This analysis reveals the stability limits of the section under idealized conditions (under linear material behavior and geometrical linear assumptions). The obtained result shows that the geometric and torsional stiffnesses of asymmetric section profiles such as ZNP100 provide significant resistance to lateral buckling. However, since this method does not take into account material and geometric nonlinearities, it gives a higher result than the critical moment capacity valid in real engineering conditions.

5.2. The nonlinear static analysis:

According to the results of the nonlinear static analysis, the critical moment capacity was determined as 10.11 kN-m. This method is a more realistic approach that includes the effects of material yielding, geometric large deformations and loading pattern. The analysis results show that the critical moment capacity obtained from buckle analysis is reduced by about 58%. This highlights the significant influence of nonlinear effects on the lateral buckling behavior and the importance of material and geometrical limitations in sections such as ZNP100.

5.3. The AISC360-22 code approach:

AISC 360-22 recommends the use of half the critical buckling stress ($F_{cr,Z} = 0.5F_{cr,U}$) of a channel (U) section profile with equivalent flange and body properties for Z section profiles. As a result of the analytical calculation using this conservative approach, the critical moment capacity of the ZNP100 section was calculated as 5.437 kN-m. The Regulation requires a lower capacity for safe design based on this approach due to the more pronounced torsional and lateral buckling effects in asymmetric Z-section profiles.

6. Conclusion

This study highlights the overly conservative nature of the design approach recommended by AISC 360-22 for asymmetric Z-shaped sections. The suggestion to use half the critical stress of an equivalent U-shaped section substantially underestimates the actual capacity of these profiles. This finding is supported by the significant discrepancies observed among analytical calculations, buckle analysis, and nonlinear static analysis results.

The buckle analysis, which does not account for material or geometric nonlinearities, is inherently limited to elastic behavior. As such, the critical moment values derived from this method represent an idealized stability threshold rather than the actual structural performance of the section. In contrast, the nonlinear static analysis incorporates material yielding and large deformation effects, providing a more realistic assessment of the beam's capacity under practical conditions. For asymmetric sections like Z-profiles, the interaction of torsional and lateral-torsional buckling effects, combined with material yielding, contributes to a notable reduction in critical moment capacity. This distinction explains why nonlinear analysis predicts significantly lower values than buckle analysis, which is confined to elastic assumptions.

Nonlinear analysis provides a sound basis for assessing the effect of material and geometric nonlinearities on critical moment capacity, while buckling analysis is mainly an elastic stability limit determination tool. In this respect, AISC 360-22's recommendation to endorse Design by Inelastic Analysis tallies more with the realistic behavior of asymmetric sections. In this section of the Code, Users' Note does state that it is conservative to base Z-profile design only on equivalent channel section properties and encourages engineers to explore advanced design methodologies.

In conclusion, the limitations of the current design approach emphasize the importance of Design by Inelastic Analysis for asymmetric sections. This approach provides a more accurate representation of actual engineering behavior by capturing material and geometric nonlinearities. For Z-shaped profiles, such methods are essential for achieving designs that are both safe and efficient, avoiding unnecessary conservatism.

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CHAPTER X

Advancing Carbon Neutrality in Construction Through Carbon Capture, Storage, Utilization and Building Information Modeling

Anıl KUL[1]

1. Introduction

Global greenhouse gas emissions contribute to rising average atmospheric temperatures and climate instability, exerting multifaceted pressures on the biosphere, economy, and society (Masson-Delmotte et al., 2019). Among these emissions, CO_2 stands out as a major greenhouse gas, primarily originating from industrial processes and energy use. International initiatives such as the Paris Agreement and the European Green Deal have established binding targets to reduce carbon emissions, encouraging nations and industries to adopt low-carbon technologies (Fetting, 2020).

The construction sector accounts for approximately 30-40% of global carbon emissions (Figure 1), both directly and indirectly, due to carbon-intensive processes such as cement and steel production, energy consumption, transportation, and construction

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activities (Xue et al., 2022). Ordinary Portland Cement (OPC) production alone contributes 5-8% of global anthropogenic CO_2 emissions (Sousa & Bogas, 2021). Within this context, the development of technologies that enhance the carbon capture capacity of construction materials plays a critical role in advancing the sector's sustainability goals.

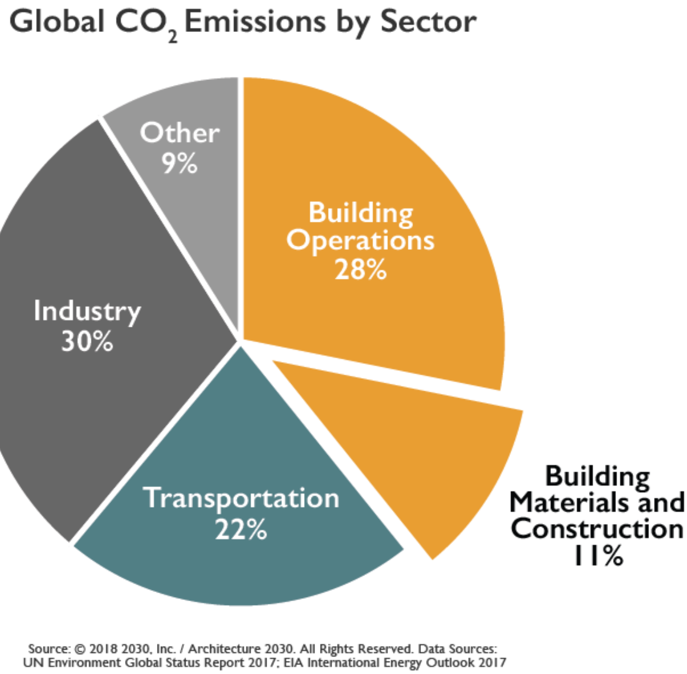

Figure 1. Contribution of the construction sector to global CO_2 emissions (Mead, 2017)

In recent years, carbon capture and storage (CCS) technologies have emerged as a promising solution to reduce carbon emissions. However, the feasibility of CCS remains a subject of debate due to its energy intensity, costs, and infrastructure requirements. Integrating carbon capture innovations into construction materials presents a cost-effective approach that can simultaneously help reduce the construction sector's carbon footprint. Various strategies have been proposed to integrate carbon capture technologies into construction materials. For instance, carbon mineralization processes enable the chemical stabilization of waste CO_2 in materials such as concrete. Additionally, research on new-generation cement types with enhanced carbon absorption capacity and low-carbon binders aims to increase carbon storage potential (Habert et al., 2020). Similarly, bio-based materials such as wood are considered sustainable options due to their ability to sequester atmospheric CO_2 through biological processes.

While CCS technologies promise significant environmental benefits, their integration with construction materials also introduces notable economic and technical advantages. For instance, by-products from carbon capture processes can enhance concrete's durability and reduce lifecycle costs through extended service life. Such dual benefits align with broader sustainability objectives, combining environmental stewardship with cost-efficiency. Nonetheless, the realization of these benefits is hindered by several challenges. The energy demands and costs associated with carbon capture, the variability in carbon absorption efficiencies across material types, and the lack of universally accepted standards for assessing carbon sequestration capacity in construction materials highlight the need for further innovation and policy support (Gálvez-Martos et al., 2021).

One potential approach to overcoming these challenges involves leveraging digital tools such as Building Information Modeling (BIM). By integrating detailed modeling of material properties, lifecycle assessments, and emissions tracking, BIM provides a platform to enhance decision-making in sustainable construction. For instance, BIM can simulate construction scenarios to optimize material selection and processes that improve carbon absorption while reducing overall emissions. Moreover, BIM facilitates improved collaboration among stakeholders, ensuring sustainable practices are consistently implemented across project phases (Santos et al., 2019).

Beyond the technical aspects, the role of policy frameworks and international collaboration cannot be overstated. Successful implementation requires not only advancements in material science but also harmonized standards, incentives for adoption, and robust monitoring systems. For example, policies promoting the use of low-carbon materials in public infrastructure projects can accelerate the transition toward a more sustainable construction industry (Allwood et al., 2012). Additionally, life-cycle assessments (LCAs) of carbon capture-enabled materials must be standardized to evaluate their long-term benefits comprehensively.

Despite these hurdles, recent progress has been encouraging. Innovations in carbon curing processes, the utilization of industrial by-products like fly ash and slag, and the exploration of advanced materials such as magnesium-based binders are reshaping the possibilities for carbon sequestration in construction (Andrew, 2018). Furthermore, the development of bio-cement and self-healing concretes offers exciting prospects for enhancing the sustainability of built environments (De Belie et al., 2021). Such advancements underscore the critical importance of integrating interdisciplinary research and stakeholder engagement to overcome barriers and scale up adoption.

This study aims to provide a comprehensive review of the carbon capture potential of construction materials, examining current technologies, material innovations, and implementation challenges. Specifically, it seeks to identify knowledge gaps, propose actionable solutions for reducing carbon emissions in the construction sector, and discuss the necessary steps toward achieving a carbon-neutral future. By bridging research findings with practical applications, this work aspires to contribute to the global effort to mitigate climate change and foster sustainable development in the built environment.

2. Capturing the Carbon at Source

The utilization of carbon capture strategies at the scale of building materials has gained momentum as a response to the construction industry's significant carbon footprint. Cement and concrete production are particularly notable for their high CO_2 emissions, stemming both from the calcination of limestone and the energy-intensive thermal processes required for clinker formation (Andrew, 2018). In light of the continuous expansion of the built environment, developing approaches that reduce embodied carbon in foundational materials has become a pressing environmental and industrial objective (Scrivener et al., 2018).

Integrating carbon capture methods directly into building material production processes, such as CO_2 mineralization, carbonation curing, and the development of novel low-carbon binders, enables manufacturers to significantly reduce net greenhouse gas emissions while enhancing product characteristics. Incorporating CO_2 into the microstructure of cementitious materials results in denser, more durable matrices, thereby extending the service life of constructed elements. Aligned with circular economy principles, this approach transforms captured CO_2 from an environmental liability into a valuable resource, promoting more sustainable raw material usage and improved resource efficiency.

CO_2 mineralization entails the transformation of carbon dioxide into stable carbonates, effectively reversing the clinker production process (Figure 2). This exothermic reaction can be strategically implemented at various stages of the cement and concrete life cycle, offering both environmental and economic advantages. By sequestering CO_2 in mineral form, this process not only reduces greenhouse gas emissions but also enhances the durability and longevity of concrete structures (Zajac et al., 2022).

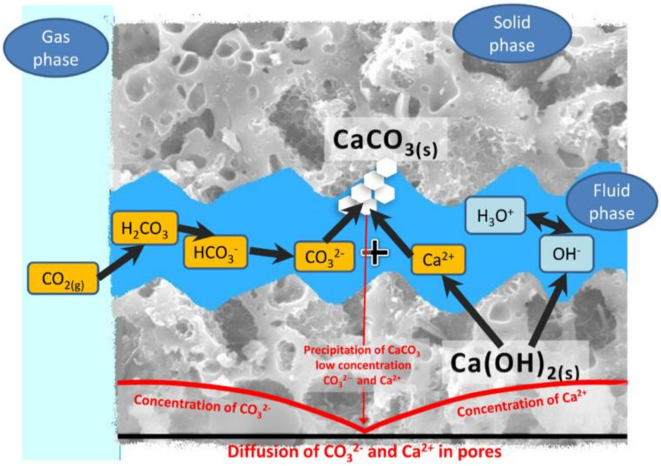

Figure 2. Carbonation mechanism (Czarnecki & Woyciechowski, 2015)

Carbonation curing, particularly when applied to magnesium-based cements, significantly improves concrete durability. Magnesium cements are characterized as low-carbon due to their lower embodied energy and inherent carbon storage capacity during their service life. The carbonation process in these cements leads to the formation of magnesium carbonates, which refine the pore structure and increase overall strength, thereby contributing to more resilient construction materials (Haque et al., 2024).

The development of low-carbon binders, such as Limestone Calcined Clay Cement (LC3), aims to reduce the clinker content in cement, thereby decreasing its carbon footprint (Fig. 3). LC3 can reduce CO_2 emissions by up to 30% compared to traditional Portland cement. This reduction is achieved by substituting a portion of the clinker with a combination of calcined clay and limestone, which not only lowers emissions but also maintains the mechanical performance and durability required for construction applications (Ijaz et al., 2024).

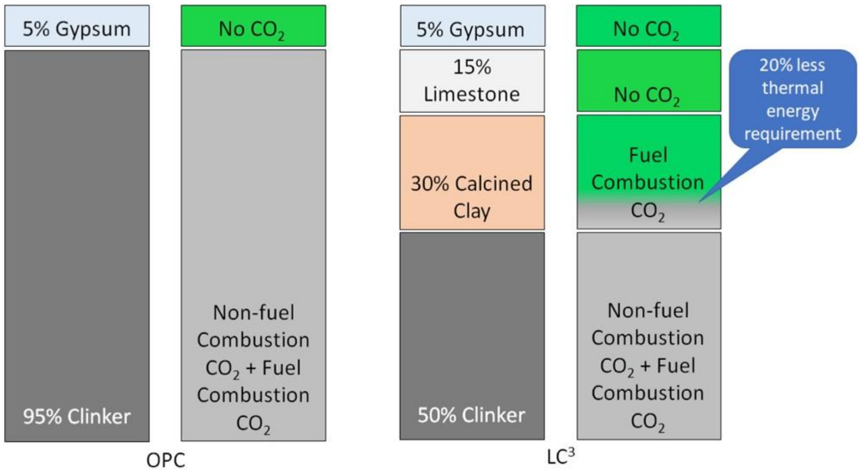

Figure 3. Limestone Calcined Clay Cement (Scrivener & Shell, 2023)

Implementing these advanced methodologies in cement and concrete production is pivotal for the construction industry's transition towards sustainability. By integrating CO_2 mineralization, carbonation curing with magnesium-based cements, and adopting low-carbon binders like LC3, the industry can achieve substantial reductions in carbon emissions while enhancing the performance and resilience of building materials. In tandem with these technical advancements, evolving policy frameworks and market dynamics increasingly favor low-carbon construction solutions. As governments introduce more stringent emission targets, enforce carbon pricing, and adjust building codes, adopting carbon capture practices can help producers preempt regulatory constraints and meet stakeholder expectations for green construction materials (European Commission, 2020). Beyond compliance, such steps enable firms to enhance their reputations and strengthen competitiveness in a market progressively oriented toward sustainable innovations (Zhang et al., 2014).

In essence, applying carbon capture within the context of building materials is not only about cutting emissions but also about enhancing material performance, optimizing resource use, and ensuring resilience against future policy changes. These converging factors highlight the strategic importance of carbon capture technologies and indicate that their ongoing refinement and commercial scaling may profoundly influence the trajectory of future low-carbon construction practices.

2.1 Passive Atmospheric Interaction and CO_2 Uptake

Carbonation processes in cement-based materials constitute a fundamental phenomenon that impacts both the chemical stability and the mechanical integrity of constructed environments, as it involves the penetration of atmospheric CO_2 into the concrete's pore network, converting calcium hydroxide ($Ca(OH)_2$) and other calcium-bearing phases into calcium carbonate ($CaCO_3$) (Neville, 1995). This reaction not only modifies pore structure and material durability but also has implications for the long-term CO_2 storage potential of cementitious matrices (Mehta & Monteiro, 2014).

Recent work has highlighted the potential of carbonation processes as a strategy for enhancing mechanical properties, increasing density, and reducing net greenhouse gas emissions (Papadakis, 2000). In contrast, traditional perspectives primarily focused on its deleterious effects—particularly the lowering of pH that may promote steel reinforcement corrosion (Basheer et al., 2001). As global construction practices strive for lower carbon footprints, understanding and controlling carbonation processes become integral to optimizing both the performance and environmental profile of concrete structures (Thomas, 2013).

Under natural service conditions, concrete is continuously exposed to atmospheric CO_2, leading to a gradual and inherently passive carbonation process. This phenomenon establishes the context for understanding the chemical and mechanical impacts of carbonation, framing both its potential benefits and challenges (Richardson, 2002). Often referred to as "natural" or "ambient" carbonation, this phenomenon occurs at relatively slow rates and is influenced by a complex interplay of factors. External parameters, such as ambient CO_2 concentration, relative humidity, and temperature, jointly shape the kinetics, while internal material characteristics like permeability, pore connectivity, microstructure, and alkalinity determine the extent and depth of carbonation fronts (Dyer, 2017). Over extended periods, even marginal concentrations of CO_2 in the surrounding environment may diffuse into the porous concrete network, reacting with calcium hydroxide ($Ca(OH)_2$) and other hydration products to form stable calcium carbonate ($CaCO_3$) phases. This gradual accumulation of carbonate minerals subtly modifies the matrix composition and the overall durability profile of the concrete (Castro et al., 2011).

Natural carbonation (Figure 4), while contributing to the densification of the matrix and potentially improving mechanical properties, has historically been viewed with apprehension due to its potential to reduce the internal pH of concrete and undermine the passive oxide layer protecting steel reinforcement (Page & Treadaway, 1982). As carbonation advances inward, the alkalinity of the pore solution declines, risking the depassivation and subsequent corrosion of embedded steel bars (Bertolini et al., 2013). This scenario can compromise the long-term structural integrity and service life of concrete elements. Consequently, considerable research has focused on mitigating the adverse effects through various strategies, including increasing concrete cover thickness, utilizing corrosion inhibitors, and incorporating supplementary cementitious materials (SCMs) to modify pore structures or buffer pH decline (Glass & Buenfeld, 2000). These approaches not only address durability concerns but also align with broader sustainability goals by enhancing resource efficiency and reducing the carbon footprint of construction practices (Scrivener & Kirkpatrick, 2008).

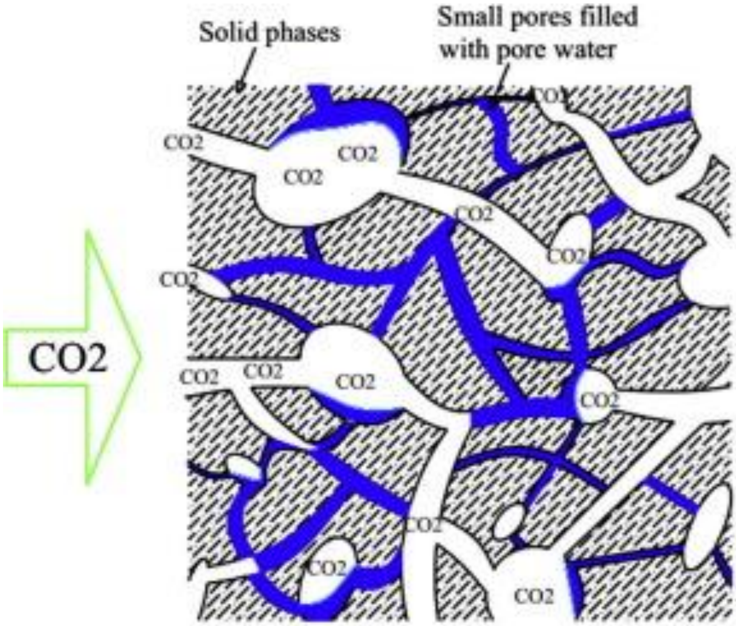

Figure 4. Natural carbonation mechanism of concrete (Šavija & Luković, 2016)

Nonetheless, recent perspectives recognize that natural carbonation does not operate solely as a risk factor. The precipitation of $CaCO_3$ within pore voids can lead to pore refinement, reducing permeability and potentially enhancing aspects of the concrete's durability. This densification effect narrows the pathways available for aggressive agents, such as chloride ions or sulfate solutions, to penetrate, potentially mitigating other durability concerns. The challenge lies in balancing the benefits of natural carbonation-induced densification against the risks of depassivating reinforcement (Teplý et al., 2010). In practice, this balance may be influenced by the specific exposure conditions, structural requirements, and performance targets set forth in codes and standards.

Ongoing research efforts seek to leverage natural carbonation's positive attributes without compromising fundamental structural reliability. Strategies include the deliberate selection of raw materials and SCMs that slow the pH decline while still permitting beneficial carbonation-induced densification. Additionally, controlling the microclimate, by managing humidity, applying surface treatments that modulate CO_2 ingress, or adjusting ventilation conditions, can help optimize the rates and depth of carbonation. Advanced mix design optimization, which tailors the mineralogical composition and pore structure of concrete, is also being explored to create a more controllable carbonation environment (Wang, 2019). Such endeavors reflect a nuanced understanding that natural carbonation, once primarily regarded as a detrimental factor, can be reinterpreted as a phenomenon with both constructive and adverse implications. By managing these dual aspects, it is possible to extend concrete longevity, reduce maintenance demands, and enhance overall sustainability within the built environment.

2.2 Mineralization of Carbon Dioxide in Cementitious Systems

The integration of CO_2 mineralization into cementitious systems has emerged as a promising strategy to reduce carbon emissions in the construction industry. This approach involves the deliberate incorporation of CO_2 at various stages of the material's lifecycle, from mixing to curing, to promote the formation of stable carbonate minerals. These carbonates serve dual functions: they enhance the structural integrity of the material and act as permanent carbon sinks, thereby improving material performance while mitigating greenhouse gas emissions (Huntzinger et al., 2009).

Recent studies have demonstrated the efficacy of CO_2 mineralization in concrete production. For instance, sequestering CO_2 during the mixing phase can improve the compressive strength of concrete by up to 10%, allowing for a reduction in cement content without compromising performance. This not only decreases the carbon footprint but also contributes to resource efficiency (Zajac et al., 2022). Another study explored the use of CO_2 as an admixture in concrete production. The findings suggest that introducing CO_2 during the mixing process leads to the formation of calcium carbonate within the cement matrix, enhancing durability and reducing permeability. This method offers a practical means of utilizing captured CO_2, transforming it into a beneficial component of the concrete (Kazmi & Prasad, 2023). Furthermore, the development of carbonation curing techniques has shown significant potential. By exposing freshly cast concrete to CO_2-rich environments, rapid carbonation occurs, resulting in the formation of stable carbonates. This process not only sequesters CO_2 but also enhances the mechanical properties of the concrete, such as increased compressive strength and reduced porosity. Carbonation curing could reduce the embodied carbon of concrete by up to 30% compared to traditional curing methods (Meesaraganda & Kazmi, 2024). In addition to laboratory research, several companies have begun to implement CO_2 mineralization technologies at scale. For example, CarbonCure Technologies injects captured CO_2 into fresh concrete during mixing, where it becomes permanently mineralized. This technique has been adopted in numerous ready-mix concrete plants, demonstrating its feasibility and effectiveness in reducing carbon emissions in real-world applications. The incorporation of supplementary cementitious materials (SCMs) further enhances the potential of CO_2 mineralization. Materials such as fly ash, slag, and silica fume not only reduce the reliance on Portland cement but also exhibit higher reactivity towards CO_2, facilitating the formation of stable carbonates. This synergy between SCMs and CO_2 mineralization contributes to both improved material properties and reduced environmental impact (Make, 2019).

In summary, the deliberate incorporation of CO_2 mineralization in cementitious systems represents a multifaceted approach to enhancing the sustainability of concrete. By forming stable carbonate minerals within the concrete matrix, this method improves structural performance and serves as a permanent carbon sequestration mechanism. Ongoing research and industrial applications continue to refine these techniques, offering promising pathways for reducing the carbon footprint of the construction industry.

2.3 Engineered Conditions for Enhanced Carbonation

To enhance the carbonation process and achieve faster reaction rates, researchers have developed techniques that increase the availability of CO_2 in the system. Known as "accelerated carbonation," this method involves exposing fresh or partially hardened cement-based materials to environments with CO_2 concentrations far above those found in the atmosphere (Figure 5). By carefully controlling parameters such as CO_2 pressure, humidity levels, and exposure duration, this approach significantly speeds up the conversion of calcium-rich hydration compounds, such as calcium hydroxide ($Ca(OH)_2$) and calcium silicate hydrate (C-S-H), into calcium carbonate ($CaCO_3$) (Bertos et al., 2004).

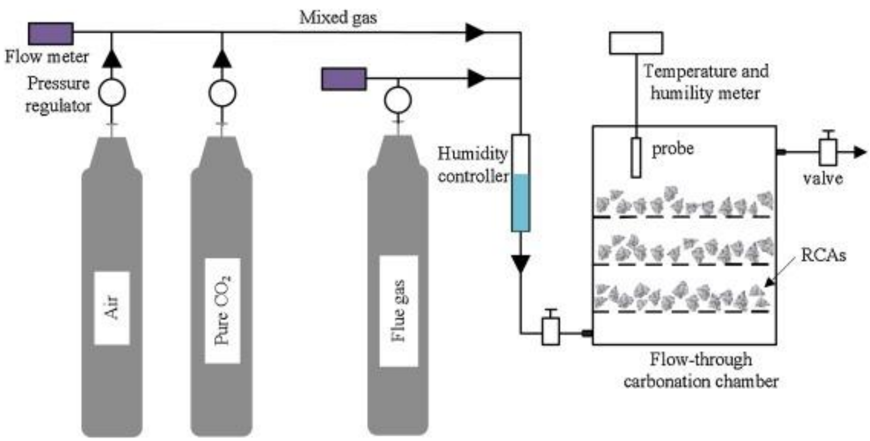

Figure 5. Accelerated carbonation mechanism (Pu et al., 2021a)

The resulting precipitation of calcium carbonate not only accelerates mineralization but also produces a more compact and durable cement matrix compared to natural carbonation processes. Research indicates that accelerated carbonation can boost compressive strength, improve microstructural density, and reduce material permeability, enhancing overall performance. For example, studies have demonstrated that high-CO_2 curing conditions can lead to rapid microstructural changes, achieving improvements in physical properties within hours instead of the weeks or months required under natural conditions (Ren et al., 2025).

A notable application of accelerated carbonation is in the production of precast concrete components such as masonry units, panels, and blocks. Under controlled conditions, these elements can be treated with CO_2 concentrations ranging from 10% to 100%, which is significantly higher than typical atmospheric levels. This process results in consistent enhancements in strength, durability, and water resistance. For instance, research has shown that precast elements treated in CO_2-rich environments exhibit superior compressive strength and lower water absorption rates compared to those cured conventionally (Kua & Tan, 2023). Moreover, accelerated carbonation effectively reduces the permeability of concrete, which is critical for long-term durability. Lower permeability limits the ingress of aggressive agents, such as chlorides, that can cause reinforcement corrosion. Experimental data suggest that chloride ion diffusion in accelerated-carbonated concrete can be reduced by more than 50%, providing better performance in harsh environmental conditions (von Greve-Dierfeld et al., 2020).

In addition to its technical benefits, accelerated carbonation offers significant environmental advantages. This method uses CO_2 emissions as a resource, capturing and embedding them within concrete's microstructure. By converting CO_2 into a stable mineral form, the process not only mitigates emissions but also supports circular economy principles by turning waste into a useful material. Estimates suggest that employing accelerated carbonation in precast concrete production could sequester substantial amounts of CO_2 per ton of cementitious material, thus contributing to carbon reduction goals (Pu et al., 2021b). Economically, the integration of accelerated carbonation into existing manufacturing setups is promising. Retrofitting traditional curing facilities with CO_2 injection systems has been found to be cost-effective, with potential financial benefits arising from material savings and reduced emissions-related penalties (Pan et al., 2012).

As the construction industry adopts more sustainable practices, accelerated carbonation is likely to play a critical role. However, challenges such as the scalability of high-CO_2 curing systems and the consistent supply of captured CO_2 need to be addressed. Further research on optimizing operational parameters and assessing the life cycle impacts of this process will be vital to its broader adoption. In summary, accelerated carbonation represents an innovative solution that improves the mechanical and durability characteristics of cementitious materials while addressing environmental concerns. Its potential to combine enhanced performance with sustainable practices positions it as a key technology for the future of concrete production.

3. BIM as a Catalyst for Carbon-Neutral Construction Practices

The construction industry's substantial carbon footprint necessitates innovative strategies to mitigate environmental impacts. Building Information Modeling (BIM) has emerged as a transformative tool, offering a digital framework to enhance the integration of carbon capture technologies within construction processes. Through its various functionalities, BIM is driving significant progress toward carbon-neutral construction (Liu et al., 2023).

BIM facilitates precise modeling of building components, enabling the simulation of diverse design scenarios to optimize material selection and placement. This capability is particularly beneficial for incorporating carbon-absorbing materials, such as carbon-sequestering concrete, into building designs (Gan et al., 2023). By utilizing BIM, designers can strategically position these materials to maximize carbon capture efficiency, thus contributing to the reduction of greenhouse gas emissions in the built environment. Moreover, BIM's role extends beyond design optimization by providing a framework for evaluating and enhancing a building's overall sustainability throughout its lifecycle, including construction, maintenance, and eventual decommissioning.

When integrated with Lifecycle Assessment (LCA) tools, BIM enables a comprehensive evaluation of a building's environmental impact across its entire lifespan. For instance, during the construction phase, BIM can pinpoint processes like cement production or steel manufacturing that have high carbon outputs, allowing for targeted mitigation strategies such as the use of alternative materials or renewable energy sources (Petri et al., 2017). Similarly, in the operational phase, BIM can help optimize energy consumption by analyzing the lifecycle performance of HVAC systems or lighting solutions (Jung et al., 2013). This integration allows stakeholders to identify carbon-intensive stages and implement carbon capture solutions at optimal points in construction and maintenance processes. Recent studies, such as those demonstrating BIM's utility in creating carbon-reducing designs for large infrastructure projects, underscore its potential to decarbonize the construction sector by providing actionable lifecycle data and evidence-based strategies (Yang et al., 2022).

In addition to its technical advantages, BIM fosters enhanced collaboration among architects, engineers, and sustainability consultants. The platform's shared digital environment facilitates seamless communication and ensures that carbon capture considerations are integrated from the project's inception. This interdisciplinary approach enables informed decision-making aligned with sustainability objectives and ensures effective coordination in implementing carbon capture technologies (Ali et al., 2022).

Despite its transformative potential, the integration of BIM with carbon capture technologies is not without challenges. One significant hurdle is the lack of standardized data protocols, which hampers the seamless sharing and analysis of carbon-related information across platforms (Lima et al., 2024). For instance, discrepancies in material databases often lead to inconsistencies in carbon footprint calculations, affecting the reliability of BIM-generated insights. Additionally, developing robust carbon accounting methodologies remains a pressing need. Current approaches are limited in their ability to comprehensively account for dynamic variables such as changing material properties over time or regional variations in carbon sequestration potential.

Ongoing research is addressing these gaps by focusing on the creation of interoperable data systems that facilitate uniform carbon assessments across projects. For example, projects leveraging open BIM standards, such as the Industry Foundation Classes (IFC), are showing promise in harmonizing data exchange (Liu et al., 2021b). Furthermore, advanced modeling techniques, including the integration of machine learning algorithms, are being explored to enhance the predictive accuracy of carbon sequestration potential within BIM environments.

Practical applications of BIM are increasingly demonstrating its transformative potential. For instance, pilot studies in urban infrastructure projects have shown how integrating real-time monitoring systems within BIM environments can dynamically adjust carbon capture strategies during both construction and operational phases. These systems utilize sensor data to track emissions in real-time, enabling immediate corrective actions, such as optimizing machinery usage or adjusting material applications to enhance carbon sequestration (Abanda et al., 2021).

Moreover, BIM has been applied to large-scale building projects to integrate renewable energy systems like solar panels and wind turbines, ensuring their placement maximizes energy efficiency while contributing to overall carbon reduction goals. For example, in a recent case study of a net-zero building, BIM was used to simulate various design scenarios, identifying optimal orientations for solar panels and airflow patterns to reduce heating and cooling demands (Garde et al., 2012).

Addressing these challenges comprehensively requires fostering industry-wide collaboration to standardize practices, investing in cutting-edge technological solutions, and establishing clear regulatory frameworks. For instance, partnerships between software developers and construction firms are driving innovations in interoperable BIM tools, while governments are increasingly mandating BIM adoption in public projects to align with sustainability targets (Chong et al., 2017). These efforts collectively underscore BIM’s critical role in achieving carbon-neutral construction practices through targeted, data-driven, and scalable solutions.

In summary, BIM stands as a cornerstone of sustainability in the construction sector. Its multifaceted capabilities, including design optimization, lifecycle assessment, and fostering interdisciplinary collaboration, highlight its critical role in integrating carbon capture technologies seamlessly into construction processes. By enabling precise modeling, comprehensive impact evaluations, and enhanced stakeholder coordination, BIM drives efforts toward decarbonizing the industry. As advancements in methodologies and standards continue to emerge, BIM's influence in achieving carbon-neutral objectives will undoubtedly become even more pronounced, offering a scalable and effective solution to the industry's environmental challenges.

4. Conclusion

In light of the extensive review presented, it is evident that integrating carbon capture technologies into construction materials provides a multi-faceted approach to addressing the environmental challenges posed by the construction industry. These technologies not only reduce carbon emissions but also enhance material performance and promote sustainability. Below are the key conclusions derived from this study:

- Carbon capture technologies, including CO_2 mineralization, carbonation curing, and the use of low-carbon binders, significantly reduce greenhouse gas emissions associated with cement and concrete production. These methods align with global sustainability goals and contribute to a circular economy.
- Techniques such as carbonation curing and accelerated carbonation improve the mechanical properties and durability of concrete by refining its microstructure and reducing

permeability. This enhances the lifespan and reduces the maintenance needs of constructed elements.

- Integrating carbon capture methods can lead to cost savings through improved resource efficiency and reduced energy consumption. Technologies like CO_2 injection during mixing and the development of low-carbon binders offer viable pathways for balancing economic and environmental objectives.
- Despite their potential, these technologies face challenges such as the need for specialized equipment, high initial costs, and limited applicability in cast-in-situ scenarios. Scaling up and optimizing these methods require significant advancements in technology and infrastructure.
- As governments impose stricter regulations on emissions, the construction industry must adopt carbon capture technologies to remain competitive and meet sustainability targets.
- BIM's comprehensive capabilities make it an indispensable tool for achieving sustainability in construction, offering transformative solutions that address environmental challenges through data-driven methodologies and interdisciplinary collaboration.

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CHAPTER XI

Building Beyond Conventions: 3D Concrete Fabrication

Anıl KUL[1]

1. Introduction

In the past decade, 3D concrete printing technology has gained significant momentum in both industrial applications and academic research as an innovative method poised to revolutionize the construction industry. This technology draws attention by offering sustainable solutions to the major problems faced by traditional construction methods (Schutter et al., 2018). In particular, it plays a critical role in the goal of reducing resource consumption and environmental impact, one of the construction industry's biggest challenges. 3D concrete printing aims to significantly increase efficiency in the sector by minimizing the amount of concrete used in construction projects and providing automated, fast, and high-precision production processes.

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With the advancement of technology, research in 3D concrete printing has focused on sustainability and productivity. These studies offer advantages both economically and environmentally, bringing a fresh breath to the construction industry. For example, thanks to this technology, waste material production is reduced, energy consumption is optimized, and the carbon footprint is minimized (Favier et al., 2018). Additionally, structural elements with complex geometries can be produced with less labor and in a shorter time. This not only reduces labor costs but also minimizes health and safety risks.

Academic research has shown that 3D concrete printing not only improves the technical aspects but also reveals the potential of this technology in terms of architectural design freedom and functionality. Specifically, the use of innovative materials and advanced software integration allows for the development of unique solutions in building design. This situation opens new horizons in modern architecture while encouraging the construction industry to evolve toward a more environmentally friendly and efficient future.

Given the increasing interest in this technology in recent years and the significant time-saving advantages offered by 3DCP, the question arises whether the full potential of this technology has been reached. However, achieving these advantages is not without cost, as it often requires introducing new challenges into concrete construction projects. This article aims to provide a comprehensive answer to this question by briefly reviewing the technology's history and current state, highlighting the key (sometimes contradictory) challenges in manufacturability and sustainability, and offering development suggestions for a more efficient, environmentally reduced future.

2. History of 3D Printing Technology

Although 3D concrete printing technology originated at Rensselaer Polytechnic Institute in the United States, its integration into the construction industry gained attention when Pegna et al.

demonstrated its applicability and potential in construction in 1997. Later, at the University of Southern California (Khoshnevis et al., 1998), a technology called Contour Crafting® (CC) enabled the automated casting of complex shapes and smooth surfaces. This marked the first large-scale applications of 3D concrete printing. However, for some time, the progress of these technologies in construction remained limited while their effectiveness continued in other manufacturing sectors. In 2009, Buswell and his team at Loughborough University developed the Conprint 3D® technology, which worked with simpler equipment and software. This innovative technology marked a significant step in the digital production of concrete structures. In 2012, ETH Zurich launched the Mesh Mold project, enabling the production of concrete structures with high geometric complexity. This project revolutionized traditional formwork systems. In 2014, Lloret and his team from ETH Zurich developed the Smart Dynamic Casting (SDC®) technology, which allowed the creation of sliding formwork around steel reinforcement. This interest accelerated research and development efforts focused on mass production. Specifically, the integration of robotic manufacturing systems and automation technologies allowed concrete structures to be produced more quickly, economically, and precisely.

In recent years, sustainability and the use of environmentally friendly materials have also been integrated into these technologies. For example, there has been a focus on 3D concrete printing with recycled materials and mix designs that provide a low carbon footprint. All these developments have promoted not only digitalization in concrete production but also greener and more efficient manufacturing processes. These innovative steps are reshaping the future of the construction industry and offering more flexible, creative, and sustainable solutions.

However, the difficulty of quality control and thus the challenges of compliance with regulations have caused 3DCP applications to focus mostly on non-structural areas. This has been a significant shortcoming that slows the industry's maturation.

Additionally, while much of the research has focused on printable material development and characterization, critical topics such as process control, reinforcement strategies, or structural design have not been sufficiently addressed, representing another barrier to progress in the sector. In order for 3DCP to establish a permanent place in the construction industry and fulfill its promises of sustainable and environmentally friendly production, academic and international research platforms have aimed to create common scientific foundations, such as classification and testing procedures, by publishing the first national standards in the U.S. (ISO, 2023) and China (T/CECS, 2020) for both fresh and hardened mechanical and durability properties.

This process, although shaped by international cooperation and scientific efforts, has progressed relatively slowly. While some countries have made progress in the creation and implementation of standards, there are doubts as to whether this process will be widely and effectively adopted on a global scale. These doubts raise important considerations about the sustainability and global positioning of 3DP in production.

3. Sustainability in Production

The adoption of digital printing in construction industry has provided transformative benefits, such as eliminating the need for formwork, creating more complex shapes, accelerating construction times, and reducing costs. However, this transformation has brought new engineering challenges. Tasks that were previously performed by formworks, such as supporting fresh concrete, now rely entirely on the material itself (Faludi et al., 2015). As a result, many 3DCP applications are still done through trial and error, leading to additional material waste (Kunnari et al., 2009). Furthermore, the formulation of materials for 3D concrete printing is crucial. The rheological properties and other fresh-state characteristics directly affect the final shape and structural integrity of printed elements, making it essential to obtain the correct material composition (Marchon et al., 2018). This is because, after mixing, the material should initially have a fluid structure to facilitate pumping and extrusion. However, once a layer is extruded, the material's hardness and strength must be high enough to preserve the layer's shape without excessive deformation under its own weight (Rehman et al., 2021). Otherwise, the risk of excessive deformation or structural collapse arises. During this process, the bond between layers is a critical factor that determines the material’s mechanical performance and durability. At this point, uncertainties stemming from material and experimental conditions are possible. Material rheology, extrusion rate, and the development rate of material properties create a direct link to strength, which is critical. The printing speed and length must be maintained within optimal values, which vary depending on the characteristics of each material.

The 3D concrete printing process involves opposing rheological requirements. High workability is needed during the pumping stage, while low workability and high thixotropy are required after extrusion. Throughout the process, a balance must be achieved between pumping, extrusion, and buildability. Concrete with low yield stress facilitates pumping but may cause shape retention issues; on the other hand, concrete with high yield stress and viscosity retains shape but makes pumping and extrusion more difficult.

Characterizing the behavior of a material across the critical time and dimensional scales of the entire 3DCP process is a complex phenomenon. For determining appropriate workability, slump tests or pressure tests for adequate strength may no longer be sufficient (Wolfs et al., 2024). In the context of digital manufacturing, concrete testing procedures have created new challenges, making it an active research topic within the academic community.

The layered structure of the 3D printing method can cause negative impacts on mechanical performance due to the interfaces formed between the layers (Feng et al., 2015). If sufficient bonding between layers is not achieved, weak areas may form, reducing the structure's strength. In particular, interfaces with low bond strength can lead to crack formation and a decrease in load-bearing capacity. These performance losses can vary depending on the material properties, printing sequence and duration, environmental conditions, and printing speed. Therefore, material modifications and process improvements that enhance bonding quality at interfaces are critical to improving the performance and reliability of 3D concrete printing technology.

Given the layer effect, using established test protocols on cast samples to determine mechanical or durability performance may be insufficient for hardened material properties. The extent of this performance degradation caused by interfaces depends on various material and process parameters, as well as differences in how layers are subjected to loads or environmental effects (Ding et al., 2020, and Panda et al., 2017). In this case, the creation of updated test protocols for 3D concrete printing (3DCP) is necessary instead of relying on traditional (destructive) testing protocols. Adding parameters such as the relationship between surface roughness and strength or moisture content and strength will yield more accurate results compared to traditional strength measurements.

The systematic and comprehensive evaluation of fresh and hardened material properties using appropriate testing methods will enable the development of more solid foundations for producing analytical or numerical simulations that can model the characteristic behavior of the 3D concrete printing (3DCP) process.

Another point is that the special material properties and printing processes of 3DPC necessitate the development of new reinforcement techniques that are not compatible with traditional reinforced concrete methods. Structural designs are targeted to be optimized to carry bending and shear loads, but existing reinforcement methods are inadequate for adapting to 3DPC printing processes, which differ significantly from traditional reinforced concrete technologies. For example, existing design and calculation methods, which are typically based on standard reinforced concrete theories, cannot fully address the specialized mechanical properties and printing processes of 3DPC structures. This limits the potential applications of 3DPC in civil engineering and highlights the need for innovative reinforcement technologies that are compatible with printing processes.

Current connection methods used in assembling 3DPC structures face limitations, particularly due to the precision required for connections and the complexity of the construction process (Diaferio et al., 2019). For example, pre-drilling holes for tensioned steel bar connections and adjusting printing paths to accommodate this design is required. Similarly, mechanical connections demand the development of specialized connection elements, and extrusion-based connections are typically limited in application because they depend on the stability of the structure itself. While traditional assembly methods can be applied to 3DPC, they fall short in controlling print accuracy and simplifying processes (Davids et al., 2022). In this context, developing safer, more efficient, and applicable assembly technologies is necessary for 3DPC to have a broader field of application.

In conclusion, while 3DPC technology holds revolutionary potential in the construction industry, it faces many technical challenges, from material properties and structural optimization to connection systems and assembly processes. Overcoming these challenges is a critical step for 3DPC to be widely adopted in a broader application area and become a standard method in the future construction industry.

4. Environmental Sustainability

3D printing technology stands out as a remarkable innovation in terms of environmental sustainability and production process efficiency. Based on layer-by-layer manufacturing methods, this technology offers a more eco-friendly alternative to traditional methods, with the potential to reduce material waste and save energy. For instance, it has been reported that 3D printers can reduce energy consumption by 41% to 64% in the production of polymer products, significantly decreasing their carbon footprint (Kreiger and Pearce, 2013). Streamlining these processes not only lowers costs but also enhances operational efficiency by reducing complexity in logistics chains (Mani et al., 2014).

This innovation is also creating profound changes in logistics and supply chains. The simplification of processes such as molding, shaping, and material transportation, which are required in traditional manufacturing methods, shortens production times and reduces the carbon footprint. Mani et al. (2014) note that layer-based manufacturing eliminates the need for complex tooling, facilitating on-site part production and providing sustainable solutions. Moreover, producing with recyclable materials offers advantages from a circular economy perspective.

However, the environmental impacts of 3D printing can vary depending on the application conditions. Kurman and Lipson (2013) point out that 3D printers used on an industrial scale can, in some cases, consume more energy than traditional methods like injection molding. Specifically, fine particle emissions during the printing of plastic materials can pose risks to both human health and the environment (Stephens et al., 2013).

Material composition is a frequently emphasized topic in academic literature (Ma et al., 2022), enabling the development of innovative solutions in structural design. Improving elements such as size, shape, and topology to optimize material use can reduce resource consumption while enhancing structural performance (Bayat et al., 2023; Gebhard et al., 2024). Traditional formwork systems, due to high costs and operational complexity, limit the implementation of such innovations. However, 3D printing technology overcomes these barriers, enabling the production of complex geometries. Nonetheless, to fully realize the effectiveness of these solutions, new design tools must be developed to align the mechanical behavior of concrete with production techniques. Today, while most 3D-printed concrete structures mimic traditional design approaches, innovative designs that take advantage of geometric flexibility are becoming more common. These designs aim to enhance strength or build pressure-resistant structures through features such as curved forms and local reinforcement elements (Bhooshan et al., 2022; Wolfs et al., 2023).

In addition to reducing material usage, extending the service life of printed structures can significantly reduce the overall environmental impact. However, a large portion of the monolithic structures produced on-site limits the principles of circular economy, such as reuse or repair. In this context, "dry connection" solutions in 3D concrete printing (3DCP) structures are emerging as a promising research area. The geometric flexibility offered by digital manufacturing allows for the creation of innovative designs with sufficient structural capacity, such as sinusoidal or tooth-like removable connections (Bischof et al., 2023; Lanwer et al., 2022).

The durability and environmental impact of printed structures are directly related to material and production process optimization. For example, high cement content not only increases production costs but also significantly raises carbon emissions. Incompatibility between layers can weaken mechanical integrity. Such issues have made quality control and material optimization a critical research area for improving the performance of printed structures and minimizing their environmental impacts (Van Tittelboom et al., 2024).

Furthermore, reinforcement is a crucial topic for the effective use of 3D-printed concrete structures. Recent developments in various reinforcement approaches (Asprone et al., 2018; Mechtcherine et al., 2021; Kloft et al., 2020) offer innovative solutions for 3DCP. Flexible and modular reinforcement systems play an important role in producing complex geometries, while the use of larger steel sections or hybrid materials may be necessary to provide sufficient structural capacity. However, the layer-based manufacturing process complicates the continuous integration of reinforcement throughout the structure. This could limit the geometric freedom of 3D printing and negatively affect structural integrity. In addition to reinforcement, the effects of new material compositions and binders on durability must be thoroughly examined. Innovative approaches have the potential to enhance structural performance while reducing environmental impact.

To better understand the environmental impacts of 3D printing technology, the application of systematic methods such as Life Cycle Assessment (LCA) is recommended. LCA analyzes the environmental impacts caused by a product throughout its entire life cycle, providing comprehensive information on energy consumption and material efficiency (EPA, 2006). Huang et al. (2015) noted that lightweight structural components produced with 3D printers provide significant energy savings compared to traditional methods, with an estimated annual energy savings potential of 173 million GJ by 2050.

In this context, the environmental sustainability of 3D printing technology greatly depends on the type of materials used, the design of production processes, and application conditions. For this technology to be used more efficiently, reducing energy consumption, increasing material efficiency, and minimizing waste generation are essential. Developing eco-friendly materials, designing production processes compatible with recycling, and promoting the widespread use of methods like LCA are key elements to advancing this field. Additionally, strategies such as integrating renewable energy sources and prioritizing biodegradable materials play a significant role in enhancing the sustainability of this technology. All of these efforts can contribute to optimizing the environmental and economic impacts of 3D printing, ensuring long-term sustainability.

5. Conclusion

3D concrete printing (3DCP) stands out as a revolutionary approach that aims to increase efficiency in construction and reduce the overall environmental footprint. Its ability to create complex geometries, eliminate or minimize formwork, and potentially save material and energy makes 3DCP an attractive alternative to conventional methods. However, many technical challenges remain, including the need for advanced rheological control, reliable layer bonding, and suitable reinforcement strategies. Additionally, the absence of universally accepted standards complicates the technology's broader acceptance.

Existing research shows that 3DCP can address significant concerns related to material waste, construction speed, and design flexibility. Nevertheless, it also indicates that improvements in material formulations, process control, and post-processing methods are critical to ensuring mechanical integrity and durability. As a result, the sector must go beyond preliminary development and embrace more systematic approaches, both experimental and theoretical, to enable 3DCP to grow into a fully mature, sustainable, and economically viable solution for modern construction needs.

6. Future Recommendations

The industry needs robust, universally recognized standards tailored to the unique characteristics of 3D-printed concrete. Current testing methods are often based on traditional casting approaches and do not capture essential features such as interlayer bonding and anisotropic strength. By developing specific testing protocols for different stages of the 3DCP process, from fresh-state workability to long-term durability, the sector can better ensure quality and build confidence in the technology.

Innovations in binder types, admixtures, and aggregate compositions are vital for optimizing both the printability and sustainability of 3D-printed concrete. Achieving a balance between fluidity (to facilitate pumping and extrusion) and shape retention (to maintain layer stability) is critical. Equally important is the implementation of real-time monitoring systems—such as embedded sensors and automated control algorithms—to adjust parameters like extrusion speed, layer thickness, and curing rates in response to actual conditions on the job site.

Traditional reinforcement methods are not always compatible with layer-by-layer construction. New reinforcement systems, including embedded fiber technologies and modular steel elements, should be tailored to the layer-based nature of 3DCP. Structural design principles must also be refined or developed to account for the anisotropy caused by interlayer interfaces. By addressing these design challenges, 3DCP can be more effectively utilized for load-bearing and large-scale structural applications.

Long-term testing of 3D-printed elements under realistic service conditions is necessary to establish confidence in their performance. Factors such as freeze-thaw cycles, chemical exposure, and dynamic loading must be thoroughly examined. Integrating lifecycle assessment (LCA) into the design and production phases can offer a systematic view of the energy savings and environmental benefits of 3DCP compared to conventional methods. This holistic approach aligns with circular economy principles and promotes sustainable construction practices.

The future of 3DCP lies in the seamless coupling of advanced robotics, artificial intelligence, and computational modeling. Automated systems can optimize printing paths, detect errors in real time, and ensure consistent layer quality. Moreover, developing reusable modular components and “dry connection” techniques can further streamline on-site assembly while extending the lifespan of printed structures. These strategies not only reduce labor costs but also open up new design possibilities by leveraging the full flexibility of digital construction.

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CHAPTER XII

Sustainability in Construction Enabled by Circular Economy Strategies with Design for Deconstruction

Anıl KUL[1]

1. Introduction

In recent years, the growing demand for housing has become a pressing global concern. This surge stems from diverse factors, such as population growth, natural disasters, pandemics, political instability, and migration trends. Although population growth rates are declining, the absolute population continues to rise, necessitating accelerated urban development. Currently, it is estimated that one in eight individuals worldwide lacks adequate housing, with forecasts suggesting that over three billion people will require suitable homes within the next decade. This demand is particularly acute in developing regions, where population growth and rapid urbanization amplify the housing crisis (UN, 2018). The migration from rural to urban areas has led to the proliferation of informal and irregular settlements, exacerbating existing challenges. Moreover, economic

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growth and rising incomes in emerging economies have heightened expectations for better living standards, further driving housing demand (Shiraishi & Sonobe, 2019). As a result, governments, policymakers, and international organizations face the formidable challenge of providing affordable, secure, and sustainable housing solutions. Meeting this demand is vital not only for improving living conditions but also for achieving broader sustainable development goals and reducing inequalities (Moghayedi et al., 2021). Addressing these challenges requires comprehensive strategies and policies to ensure equitable access to adequate housing. However, the current linear economic model in the construction sector raises concerns about the extensive material, energy, and time resources required to meet this escalating demand.

The traditional linear economic model is characterized by a "take, make, dispose" approach, which involves extracting raw materials, producing goods, and discarding them after use. This model is inherently unsustainable, as it promotes excessive consumption of finite resources and generates significant waste and emissions [7]. Globally, over 12 billion tonnes of waste are produced annually, and projections suggest that continuing with traditional economic practices could result in 167 gigatonnes of material usage by 2060, 20% of which would become waste. This equates to an average daily material usage of 45 kg per person by 2060, intensifying environmental challenges (OECD, 2021a;2021b).

The climate crisis underscores the urgency of transforming how raw materials are utilized and how manufacturing and marketing activities are conducted (UNEP, 2020). Unfortunately, the construction industry, which accounts for 40% of global natural resource consumption and 25% of global waste production, has not effectively addressed issues related to resource use, waste generation, carbon emissions, and energy consumption. This sector is the third-largest generator of daily waste per capita, producing approximately 1.70 kg per person (Kaza et al., 2018). Moreover, the building sector is responsible for 40% of energy consumption and 50% of greenhouse gas emissions globally (Chau et al., 2015).

Despite some progress in recycling and reusing materials, the global circularity rate remains low, around 9%, with nearly 60% of materials losing their value due to the lack of circular economy practices (Ossio et al., 2023).

Transitioning to a circular economy and mitigating the environmental impacts of the construction industry require changes across every phase of a building's life cycle and throughout the entire value chain. To address these challenges, Design for X (DfX) methodologies have gained traction in research, innovation, and manufacturing. These methodologies focus on aligning product and process design with specific goals such as cost optimization and quality enhancement. Within the construction sector, Design for Deconstruction (DfD) has emerged as a specialized approach. By integrating principles of the circular economy, DfD aims to maximize the advantages of modular construction techniques while minimizing inefficiencies and waste.

This study explores the DfD methodology in depth, emphasizing its potential to embed sustainability into the construction sector at a modular level. By advancing theoretical knowledge and understanding of sustainability's critical impacts, this research seeks to provide valuable insights for professionals striving to implement sustainable construction practices. Ultimately, this work aims to contribute meaningfully to societal progress by fostering more efficient and responsible construction approaches.

2. Design for Deconstruction (DfD) Insights

Design for Deconstruction (DfD) embodies principles tailored to product design, focusing on the eventual disassembly and disposal of components. This approach integrates strategies to dismantle manufactured items, facilitating the reuse of their parts at a later stage (Bogue, 2007). As a cornerstone of circular lifecycle methodologies, DfD aligns closely with the "reverse construction" principle, which encompasses processes for dismantling building elements to reintegrate them into reuse and recycling cycles

(Kanters, 2018). Figure 1 illustrates the sequential stages involved in the DfD principle.

DfD offers a range of options, including dismantling structures, processing materials, repurposing elements into alternative constructions, reusing components, and even relocating entire structures (Roxas et al., 2023). Implementing these alternatives promotes a circular construction framework, reducing the need for entirely new facilities while adhering to principles of responsible consumption, waste minimization, and sustainability.

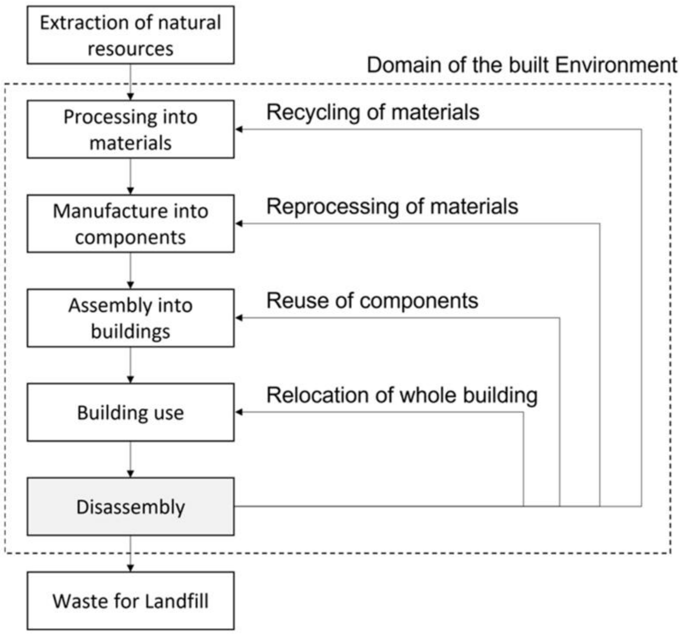

Figure 1. Stages of the DfD (Cho et al., 2022)

DfD lacks a universally recognized standard or guideline. Existing studies and limited professional recommendations typically focus on general design and construction principles compatible with DfD rather than offering explicit definitions or approaches. Research has primarily evaluated DfD within the context of collaboration and competencies that support designs involving structural frameworks, concrete and steel phases, connection elements, and construction/deconstruction workflows. The relatively recent introduction of DfD in the construction industry is a significant factor contributing to this gap. Table 1 summarizes perspectives and benefits of DfD guidelines as presented in limited studies.

Table 1. Perspectives of DfD guidelines and their benefits

	Guidelines	References
Design of the Construction	Designing a building as a multi-layered structure with individual life cycles	I-II-III
	Formulating a plan for deconstruction and waste management	III
	Minimization the number and variety of building components through modularization and standardization	I-II-III
	Utilization of off-site construction and prefabrication techniques to the building process	I-II-III-IV
	Incorporation of lightweight components	I-II-III
	Utilization accessible technical installations within an open building design	II-III
	Designing the interchangeability of building components for flexibility and ease of maintenance	III
	Incorporation of a structural grid	II-III
	Establishing a database for materials, parts, and construction data	I-II
	Integration of deconstruction type and technique within the design phase	IV
	The provision of comprehensive documentation pertaining to the designs, schematics, and images	I

Design of Components-Connections	Use of reusable, environmentally safe, non-hazardous materials	I-II-III-VI
	Use of simple, ease-to-remove connections	IV
	Use of mechanical, dry connections	II-III-VI
	Reduction in the quantity/variety of connections	II-III-V
	Considering the design to be long-lasting and durable	II-III-V
	Use of accessible components and connections	IV
	Avoid to use the composites and different types of materials	I-II-III-VI
	Application without post-manufacturing treatment	II-III
	Replacement components for maintenance	II
	Optimization of element size	II
	Identification of lifespan of each materials/components	III
	End-of-life performance determination	III
Determination of Deconstruction Details	Instructions for materials to undergo reuse and recycling	I-III
	Identification of components within the building system allocated for deconstruction	II
	Consideration of simultaneous disassembly within the design process	II
	Facilitating easy access to the entire building	II

Note: I: (Ahn et al., 2022); II: (Bertino et al., 2021); III: (Crowther, 2005); IV: (Akinade et al., 2017); V: (Carvalho Machado et al., 2018) ; VI: (Lu, 2024)

3. Sustainability Aspects of Design for Deconstruction

The construction sector predominantly follows a linear economic model characterized by a consumption-production framework that disregards the finite nature of raw materials (Kuehlen et al., 2014). This model's widespread use continues to raise sustainability concerns. Numerous studies have demonstrated that traditional construction approaches fall short in addressing environmental challenges. Conventional methods rely extensively on large quantities of materials such as concrete, scaffolding, formwork, walls, and roof elements, which result in substantial waste generation. Furthermore, inefficiencies in these methods often arise due to worker productivity issues, leading to delays and additional waste (Nurhendi et al., 2022). As a result, researchers and industry practitioners are increasingly turning towards innovative methods aligned with circular economy principles. Particularly in the context of rapid production structures, a growing focus on sustainability and environmental awareness has accelerated the integration of methodologies such as Design for Deconstruction (DfD), which incorporate circular economy ideals into modern construction techniques (Zhao et al., 2018). This shift underscores the transformative potential of DfD in revolutionizing construction workflows and mitigating adverse impacts.

Producing high-quality, cost-effective products quickly while embedding sustainability into their design is vital for meeting environmental standards. Achieving this involves minimizing the number of parts, operations, assembly, and production times, as well as selecting environmentally friendly materials and incorporating reusable connection designs. DfD has emerged as a highly effective methodology in this regard, as it emphasizes thorough planning and optimization during the design phase. This approach typically reduces waste, product complexity, costs, and production time (Boothroyd et al., 2010).

The persistent use of demolition-based practices exacerbates waste generation, increases raw material consumption, and inflates costs. Adopting DfD represents a critical step toward breaking away from these traditional practices. DfD holds the promise of becoming a cornerstone for the construction sector, which struggles under the weight of the linear economic model, by curbing unsustainable consumption-production cycles and reducing waste generation. Reuse strategies inherent to DfD aim to repurpose old structural components directly, thus lowering the demand for raw materials, minimizing production processes, and cutting waste disposal needs. For instance, concrete components are particularly suitable for reuse due to their longer service life compared to other materials (Ong et al., 2013). However, traditional connection designs often obstruct component reuse. To address this, DfD incorporates design concepts during the pre-production phase that facilitate dismantling and repair.

While significant research has focused on recycling solutions for building materials, scaling these efforts to the structural element level remains a critical challenge. Implementing DfD can drive circularity and foster sustainable development. By simultaneously minimizing deconstructible components, utilizing lightweight materials, and employing efficient systems such as prefabrication, DfD reduces costs while enhancing quality (Bertino et al., 2021). Additionally, integrating DfD principles into the design process has proven to be a practical strategy for making asset lifecycles circular and minimizing waste.

4. Adoption of DfD in Construction Industry

DfD promotes sustainability by prioritizing dismantling processes at the end of a product's lifecycle, enabling reusability and recyclability as alternatives to traditional demolition. By incorporating dismantling procedures into the pre-construction planning phase, environmentally friendly and cost-efficient outcomes can be achieved. This forward-thinking approach integrates seamlessly with contemporary construction practices, encouraging the continuous reuse of production elements across multiple systems, which significantly aids in resource conservation (Akanbi et al., 2019). The escalating threats posed by environmental pollution and the overexploitation of natural resources underscore the urgency for sustainable practices. Standardization, a hallmark of off-site production methods, is achieved through environmentally conscious techniques (Tafesse et al., 2022). However, the broader environmental implications of prefabrication methods must be evaluated, necessitating the development of robust policies and standards. The construction sector's gradual shift towards sustainable methodologies aligns with DfD's cradle-to-cradle framework, which aims to minimize environmental impact by fostering designs that support future structural adaptability and extend building lifespans through dismantling-friendly features (Lu et al., 2021).

DfD enhances the potential for reuse and recycling within the construction industry's value chain, positioning a circular economy as a practical solution for sustainability. Transitioning to a circular economy not only curtails waste generation but also promotes resource conservation and the proliferation of eco-friendly products. Simplified design strategies, the use of adaptable components, and comprehensive disassembly instructions are vital for the successful implementation of DfD. Efficient application of DfD in the construction sector can eliminate the generation of construction and demolition waste (CDW) during planning stages (Crowther, 2005). From an environmental standpoint, this approach ranks among the most effective construction waste management techniques. Structures undergoing deconstruction instead of demolition yield reusable structural elements for new construction projects (Ding et al., 2016). Additionally, by supporting sustainable development, DfD reduces waste, improves resource efficiency, and offers a more adaptable construction methodology, enabling existing structures to serve new purposes and diminishing the demand for new buildings (ISO, 2020).

Central to the DfD methodology is the design of modular and reconfigurable building components, with the development of supportive structural connections playing a pivotal role. Several DfD-specific connection types differentiate these structures from traditional prefabricated buildings, such as bolted end-plate connections (Ong et al., 2013), cast-in-place concrete connections

(Xiao et al., 2017), and embedded steel connections (Ding et al., 2018). Research into suitable materials for deconstructable structural components highlights their unique attributes and implications for sustainability. For example, Broniewicz and Broniewicz (2020) identified steel as an ideal material due to its sustainability, low waste production, reusability, and compatibility with dry construction methods. Similarly, Lu et al. (2021) proposed DfD rules for cold-formed steel structures, emphasizing ease of disassembly, increased reuse potential, and enhanced safety. Tingley (2012) explored the applicability of DfD for wood, steel, and concrete structures, noting that wood's simple construction techniques and standard dimensions make it highly suitable for deconstruction. Lime mortar in wall construction allows bricks and blocks to be reused, while cement mortar can complicate this process. Steel is generally recyclable, though its reuse depends on quality and connection methods; additional fireproofing can hinder deconstruction.

While these materials and systems may entail significant costs and labor, sustainable construction techniques must balance affordability with the accessibility of materials. Concrete, a cornerstone of traditional construction, is notable for its cost-effectiveness, widespread availability, and versatility, making it a valuable material for DfD applications.

5. Case Studies for DfD in Construction Industry

Research on integrating DfD principles has yielded significant findings. Wang et al. (2020) applied DfD principles to structural components and examined the performance of prefabricated beams and connection elements (Figure 2). Their analysis revealed that load-deflection curves for demountable systems demonstrated ductile behavior with minimal or no loss in post-yield strength. Under maximum operational conditions, beams exhibited slight non-linearity, likely attributed to beam bending, minor displacement of steel components, and joint elements. Despite this, demountable composite beams were deemed reusable with negligible risk of yielding during their lifespan.

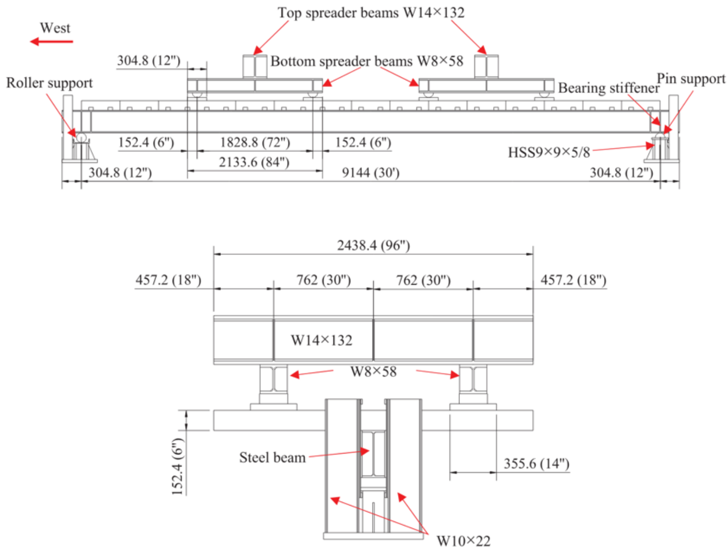

Figure 2. Deconstructable composite beam plan (Wang et al., 2020)

Ding et al. (2020) introduced a novel beam-to-column connection designed in line with DfD criteria and assessed its seismic performance (Figure 3). Experimental results demonstrated that the proposed concrete beam-column joints incorporating DfD connections sustained sufficient strength under earthquake-induced stresses. These frame joints displayed favorable ductility characteristics under cyclic loading. By enhancing the continuity of longitudinal reinforcements, these joints maintained moment capacities similar to reference specimens. The investigation highlighted their easy disassembly and reassembly during construction, facilitated by reduced dependency on in-situ concrete. Performance evaluations of reconstructed specimens confirmed that the new joints achieved desirable structural outcomes.

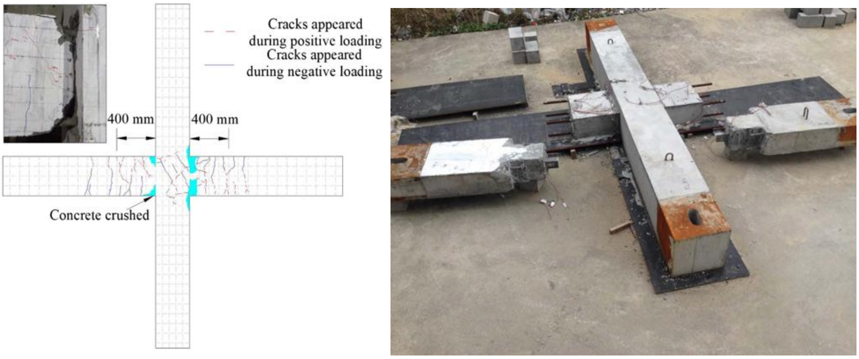

Figure 3 Design of beam-column frame joint (Ding et al., 2020)

Xiao et al. (2020) explored the impact of coarse aggregate materials on connections, finding that performance remained within acceptable limits despite minimal effects. Korkmaz and Tankut (2005) evaluated dismantling strategies for concrete joints and identified hybrid steel joints as the most suitable for implementing DfD principles in concrete structures. This approach ensures reinforcement continuity without disrupting complex reinforcement within the joint core. Leso et al. (2018) and other researchers developed wood-steel connection designs aimed at reducing concrete use while embracing recyclable materials in alignment with comprehensive DfD principles. However, they emphasized the need for further studies to refine predictions about structures designed for dismantling, especially those expected to remain operational for approximately 60 years.

A noteworthy study by Xia et al. (2020) addressed circular economy concerns, demonstrating that implementing DfD principles yields 1.8 to 2.8 times greater environmental benefits compared to conventional structures. Leising et al. (2018) emphasized designing buildings with a focus on both operational and end-of-life phases, ensuring components can re-enter the recycling market and be reused where needed to enable circularity. Ortlepp et al. (2016) compared the environmental impacts of constructions using dry versus wet connections and found that dry connections significantly increased material recyclability. Eckelman et al. (2018) evaluated a novel floor system and revealed that reusing DfD floor components reduced the environmental impact of conventional configurations by 70% when reused three times. Tingley and Davison (2012) proposed a methodology to assess life cycle impacts of DfD-oriented buildings, ensuring that environmental considerations are evenly prioritized throughout a component's lifecycle.

5. Conclusion

This study comprehensively examines the fundamental concepts, historical evolution, advantages, implementation, and influence of Design for Deconstruction (DfD) in promoting sustainable development in the construction sector. To achieve this, an extensive literature review was conducted, providing a detailed overview of the current status of techniques in the field. Process flows, fundamental principles, existing guidelines were examined, and research gaps were assessed. In addition to the theoretical review, this paper also concentrates on the detailed examination of various case studies related to DfD in construction. The principal conclusions drawn from this study are outlined below:

DfD principles center around designing components that can be reintegrated into the system at the end of their service life for the same or different purposes. These principles emphasize creating designs that allow for easy disassembly and rapid reassembly while maximizing reusability.

More comprehensive studies are needed to integrate DfD approaches, which are critical for supporting sustainable development and the circular economy, into the construction sector. Conducting studies that synthesize DfD systems, maximize the benefits derived from them, and explore their effects compared to traditional construction techniques are essential steps towards the widespread adoption of these strategies.

To fully realize the potential of DfD, future research should prioritize the synthesis of systems and the maximization of associated benefits. This requires a holistic approach that considers the entire lifecycle of built assets, from design and fabrication to operation and deconstruction. By systematically identifying synergies and trade-offs within DfD strategies, researchers can develop integrated frameworks that optimize resource efficiency, cost-effectiveness, and environmental performance throughout the construction lifecycle.

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CHAPTER XIII

Numerical Simulations of Wall Configurations for Waste-based Sustainable 3D Printing and Thermal Insulation

Anıl KUL[1]

1. Introduction

The rapid growth of the global population, expected to reach nearly 10 billion by 2050, has led to an increasing demand for housing, estimated at 20 million new homes annually (Najafi & Khanbilvardi, 2019). Furthermore, the European Commission has reported that buildings account for 40% of energy consumption and 36% of CO_2 emissions in the EU, making them the largest energy consumer in Europe (EU Comission, 2019). Similarly, the United Nations projects that the global population could reach 10.9 billion by 2100, further amplifying housing demands (UN, 2019). Combined with the significant environmental impact of building operations and the high energy consumption from urban services,

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these factors underscore the urgent need to enhance efficiency and sustainability in the construction industry.

One promising approach to these challenges is 3D concrete printing, which offers precise, layered construction while significantly reducing costs, time, and material waste. It reduces waste by up to 60%, labor costs by 80%, and production times by 70% (Zhang et al., 2019), while promoting sustainable design aligned with circular economy principles. This innovative technology also provides opportunities to improve thermal performance, making it an essential tool for addressing building energy efficiency and reducing greenhouse gas emissions.

As the global population grows, the energy spent on heating and cooling in residential areas is increasing at a significant pace. Buildings are major consumers of electricity worldwide, with climate control systems (heating, ventilation, and air conditioning) accounting for approximately 60% of this consumption. Globally, buildings are responsible for nearly 40% of total annual energy use, with a significant portion of this energy attributed to cooling needs in hot climates (González-Torres, 2022). This reliance on energy-intensive systems creates financial burdens for nations and contributes to substantial environmental emissions. One of the primary reasons for high energy consumption in buildings is their insufficient thermal storage capacity, which prevents effective resistance to heat flow caused by temperature differences between indoor and outdoor spaces. Energy losses through building envelopes make sustainable climate control processes challenging and substantially increase energy demands. Therefore, developing construction materials with high thermal insulation properties is crucial for improving energy efficiency and sustainability in buildings.

The thermal performance of exterior walls significantly affects a building's energy consumption, efficiency, and greenhouse gas emissions (Schiavoni et al., 2016). Effective insulation materials are essential for reducing fossil fuel use and CO_2 emissions. Studies indicate that thermal insulation plays a crucial role in lowering energy demands, especially in heating-dominated climates. However, challenges remain, particularly for intermittently used buildings where higher thermal mass can slow temperature adjustments and increase preheating or precooling demands (Verbeke & Audenaert, 2018). Excessive insulation can also restrict heat dissipation, raising cooling needs. Designing energy-efficient buildings requires balancing indoor thermal conditions with optimized insulation properties (Lee et al., 2017).

3D concrete printing emerges as a transformative solution to these challenges, offering innovative approaches to optimize thermal performance, durability, and energy efficiency in buildings. By integrating advanced designs and systems, this technology addresses critical issues while aligning with environmental sustainability goals. However, concerns persist regarding the sustainability of materials used in its processes, as current options often fall short in meeting environmental benchmarks. Most current mixtures rely heavily on ordinary Portland cement (OPC), which accounts for 15–45% of the total mixture and is associated with high carbon emissions and energy-intensive production processes (Zhang et al., 2021). While alkali-activated systems have been proposed as a viable alternative, their reliance on industrial by-products such as fly ash and ground granulated blast furnace slag (GGBS) presents additional challenges. These materials are becoming increasingly scarce, with diminishing supplies of fly ash in Europe and high demand for GGBS in the cement industry. This scarcity underscores the need for abundant, sustainable precursors to support the broader adoption of 3D concrete printing.

Studies have highlighted the promising role of 3D printing in enhancing thermal insulation properties. A key advantage of this technology lies in its ability to produce complex geometries, facilitating biomimetic designs inspired by ecosystems and biological structures (Du Plessis, 2019). These designs merge natural efficiency with advanced manufacturing to create lightweight, energy-efficient, and thermally optimized materials. For instance, honeycomb and wood-like cellular structures achieve thermal conductivities as low as 0.05 W/(mK), demonstrating their potential for sustainable construction (Panda, 2018; Kam et al., 2019). Innovative materials like aerogels have also been integrated into 3D-printed composites, enhancing thermal insulation while maintaining low conductivity. Additives such as air bubbles improve acoustic and thermal properties, making them suitable for on-site 3D printing. Similarly, foamed concrete and geopolymer matrices with thermal conductivities between 0.125 and 0.25 W/(mK) balance insulation with mechanical strength (Baghban, 2019; Falliano et al., 2018). The thermal insulation performance of geopolymer materials has been extensively documented in the literature. Liu et al. (2013) examined foam geopolymer concretes enriched with natural materials like palm kernel shell, reporting densities ranging from 1291 to 1794 kg/m^3 and thermal conductivities between 0.47 and 0.58 W/mK, highlighting their insulation potential. Similarly, Feng et al. (2015) optimized porous geopolymers using H_2O_2 as a foaming agent, achieving thermal conductivities of 0.0816–0.0902 W/mK while increasing porosity to 76–79%. Medri et al. (2015) investigated vermiculite-based geopolymer panels, reporting thermal conductivities between 0.178 and 0.189 W/mK, and emphasizing their fire resistance and insulation capabilities. Zhang et al. (2015) evaluated foam geopolymer concretes with slag replacements, achieving thermal conductivities of 0.15–0.48 W/mK and compressive strengths up to 48 MPa, demonstrating their durability under high temperatures. Bai et al. (2016) explored foam geopolymer materials incorporating H_2O_2 and surfactants, achieving porosities of 74–87%, thermal conductivities of 0.09–0.16 W/mK, and compressive strengths between 0.3 and 4.4 MPa.

Research into 3D-printed materials underscores their potential for thermal insulation. Gomaa et al. (2019) examined clay-based materials reinforced with organic fibers, demonstrating the feasibility of 3D-printed insulating solutions despite conventional methods yielding better performance. Similarly, Falliano et al. (2019) showed that foam concrete with smaller air bubbles achieved superior thermal performance and compressive strength at higher densities. Chung et al. (2016) highlighted the role of anisotropic voids, revealing their significant influence on thermal conductivity and stiffness through 3D-printed samples. Alghamdi and Neithalath (2019) developed geopolymer foam matrices using fly ash, achieving thermal conductivities of 0.15–0.25 W/mK with porosities between 55% and 75%. Huiskes et al. (2016) demonstrated the benefits of optimized aggregate size for thermal insulation, while Huang et al. (2018) integrated aerogels with geopolymers, achieving conductivities as low as 0.048 W/mK. Furthermore, Zenabou et al. (2019) improved thermal stability and refractory properties by substituting metakaolin with metaboxite and metatalk, maintaining low thermal conductivities and enhancing performance at temperatures up to 1000°C.

Beyond materials, 3D printing has enabled system-level applications, including modular green walls and facades that regulate indoor climates. Panels combining air voids and phase-change materials address thermal bridging and condensation issues, while projects like the YHNOVA house in France demonstrate the scalability of these solutions. Constructed in just 54 hours, the house integrates concrete and polyurethane-foam walls to provide both structural and thermal benefits (Subrin et al., 2018).

Leveraging advancements in 3D printing, this study aimed to address the challenges of material scarcity, environmental sustainability, and thermal efficiency in construction. A novel 3D-printable alkali-activated material was developed, integrating brick waste as the aggregate phase to mitigate resource limitations and reduce environmental impact. Numerical simulations were conducted to evaluate the material's thermal performance across various wall configurations. The findings reveal its potential to significantly enhance energy efficiency, lower greenhouse gas emissions, and support the integration of recycled materials into innovative construction practices, paving the way for a more sustainable built environment.

2. Materials and Methodology

The 3D-printable thermal insulation material used in this study was developed based on the mixture designs of alkali-activated, brick waste-based systems previously investigated by Kul et al. (2024a). The mixture design followed several key considerations: Brick waste was utilized as both the primary binder and aggregate phases. Specifically, fractions smaller than 100 µm were incorporated into the binder phase, while those between 100 µm and 2 mm were used in the aggregate phase. Kaolin clay was added to ensure smooth extrusion through the 3D printer nozzle, owing to its excellent flow properties, such as shear-thinning and low viscosity (Sun et al., 2018), as well as its widespread availability in various regions (Murray et al., 2000). Similarly, limestone, a cost-effective and abundant material (Kanagaraj et al., 2023), was included to enhance the reactivity of the alkali-activated system by providing nucleation sites (Long et al., 2021), improving workability, and extending open times (Bayiha et al., 2019). Ground granulated blast furnace slag (GGBS) was incorporated to promote buildability, a critical parameter for 3D printing, by supporting the formation of gel structures like CSH and CASH, which also enhance mechanical performance (Kul et al., 2024b).

To further investigate the effects of aggregate content on fresh, mechanical, and thermal properties, a series of mixtures was developed. The primary approach involved incrementally increasing the brick aggregate content, excluding alkali activators, from 50% to 80% by weight of the total dry material at equal intervals. The compositions of these mixtures are presented in Table 1. By keeping the water-to-binder ratio constant, the direct impact of aggregate content on fresh properties was assessed. The mixtures were produced by homogenously mixing the precursor phase, aggregate phase, and alkali activators in their dry state, followed by the addition of mixing water to the system. This method ensured the production of one-part mixtures.

To evaluate the suitability of fresh state properties for 3D printing, flowability and buildability tests were conducted at intervals of 0, 30, 60, and 90 minutes, starting immediately after mixing completion (with the initial measurement taken at 5 minutes, defined as 0 minutes). The flowability test adhered to the ASTM C1437 (2020) standard, and the flowability index was determined by measuring the relative changes in horizontal spread. For buildability, a mini-slump test was performed using the same setup with a 600 g load, and vertical deformations of the mixtures were calculated to assess performance (Figure 1). Additionally, compressive strength tests were conducted on 40 mm cubic specimens with a loading rate of 2400 N/s, while 3-point flexural strength tests utilized 40x40x160 mm prismatic specimens at a loading rate of 50 N/s. The measurement of the thermal conductivity value required for the numerical modeling of the mixture was carried out using the ISOMET Heat Transfer Analyzer device. To test the 3D printability of the selected mixture based on the conducted tests, a wall printing process was performed using a 3D concrete printer integrated with ABB Robotics' IRB 1200 robotic arm.

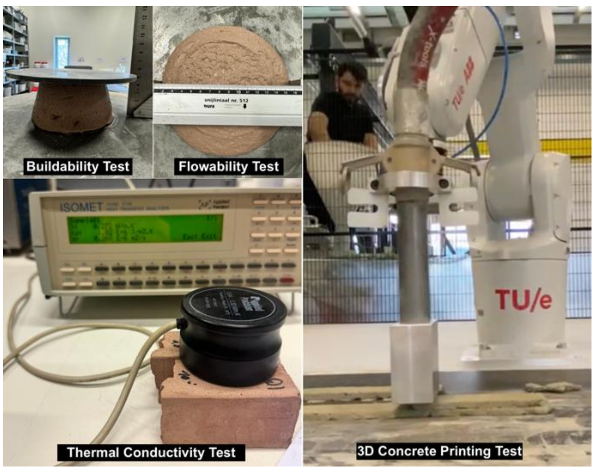

Figure 1. Samples and testing equipments

Table 1. Mixture proportions of alkali-activated mixtures

	Mixture Proportions (kg)							
	Limestone	Slag	Kaolin Clay	Brick Powder	Brick Aggregate	NaOH	Na_2SiO_3	Water
BA0.5	220	220	220	270	945	37.2	37.2	650
BA0.6	175	175	175	225	1125	30	30	650
BA0.7	130	130	130	180	1305	22.8	22.8	650
BA0.8	80	80	80	130	1505	14.8	14.8	650

3. Results and Discussions

Figure 2 illustrates the influence of varying aggregate proportions (BA0.5, BA0.6, BA0.7, BA0.8) on the flowability, buildability, and strength properties of 3D-printable materials. Flowability decreases progressively over time (from 0 to 90 minutes) across all mixtures, with lower aggregate content (e.g., BA0.5) exhibiting the highest initial workability (1.65 normalized value at 0 minutes), followed by BA0.6 (1.62), BA0.7 (1.56), and BA0.8 (1.51). This decline can be attributed to two main factors: the ongoing hydration reactions that progressively increase the viscosity of the paste and the evaporation of free water, which reduces the mixture’s fluidity (Han et al., 2024). Additionally, mixtures with higher aggregate content, such as BA0.8, experience a more pronounced reduction in flowability due to increased particle friction and the limited availability of paste to facilitate smooth movement (Assaad, 2017). These dynamics suggest a delicate balance between aggregate content and paste volume is essential for maintaining consistent extrudability. Buildability, conversely, improves with time as the material begins to set, enhancing its structural integrity. This improvement is particularly noticeable in higher aggregate mixtures, such as BA0.8, which demonstrate superior shape retention and resistance to deformation (0.94 normalized value at 90 minutes). This behavior is likely due to the increased interparticle friction and reduced paste content in these mixtures, which contribute to better stability under external loads. As a result, higher aggregate mixtures are highly suitable for structural components where maintaining geometric precision and resisting collapse under applied loads are critical.

Compressive strength increases significantly with curing time, ranging from 2.67 MPa (BA0.8 at 3 days) to 9.67 MPa (BA0.5 at 28 days). This trend highlights the ongoing hydration and pozzolanic reactions in the matrix, which contribute to the development of denser and stronger gel phases over time. Higher aggregate content (e.g., BA0.8) provides additional mechanical interlocking and a greater load-bearing capacity, resulting in markedly higher compressive strength compared to lower aggregate mixtures (e.g., BA0.5). Similarly, flexural strength follows a consistent pattern, increasing from 1.36 MPa (BA0.8) to 3.99 MPa (BA0.5) over the 28-day curing period. While flexural strength is primarily influenced by the matrix's tensile properties and the bonding quality of aggregates, the effect of aggregate content is less pronounced compared to compressive strength.

While higher aggregate content significantly enhances buildability, it simultaneously leads to a marked reduction in flowability and mechanical strength (Liu et al., 2021). This dual effect necessitates a strategic approach to material selection based on application-specific requirements. For intricate designs or structures requiring fine detailing, where extrudability is critical, mixtures with lower aggregate content, such as BA0.5, are more suitable due to their superior flowability and ease of application. Conversely, for structural elements that demand greater strength and stability, such as load-bearing walls or beams, higher aggregate mixtures like BA0.8 are ideal as they provide improved resistance to deformation and enhanced mechanical performance. The balance between flowability and buildability must be meticulously assessed, taking into account factors such as print geometry, curing conditions, and load requirements to ensure optimal material performance across diverse construction scenarios (Dwivedi, 2024).

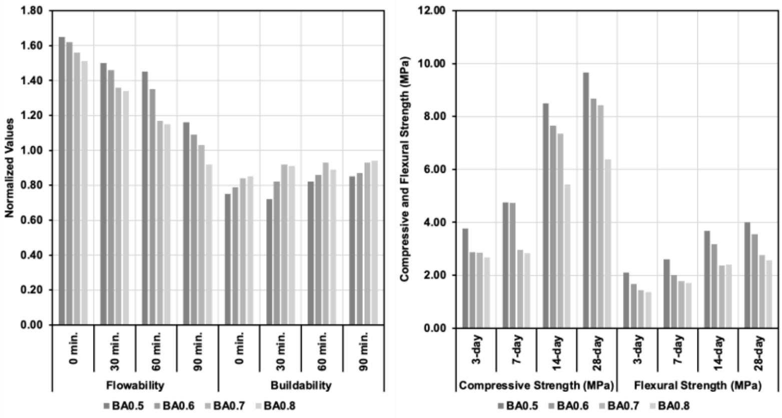

Figure 2. Fresh state properties and mechanical performances of alkali-activated mixtures

The results of mechanical tests and thermal conductivity analyses conducted on 28-day-old specimens are presented in Figure 3. Overall, the findings suggest an inverse relationship between mechanical properties and thermal conductivity performance. As previously mentioned, the increasing aggregate content negatively impacts the mechanical properties while leading to a decrease in thermal conductivity. This can be attributed to the higher amount of brick waste aggregate reducing the matrix density. Furthermore, during the mixing process, the aggregate traps water within its structure, which later escapes over time, resulting in the formation of more voids within the matrix (Qi et al., 2023; Padmini et al., 2002).

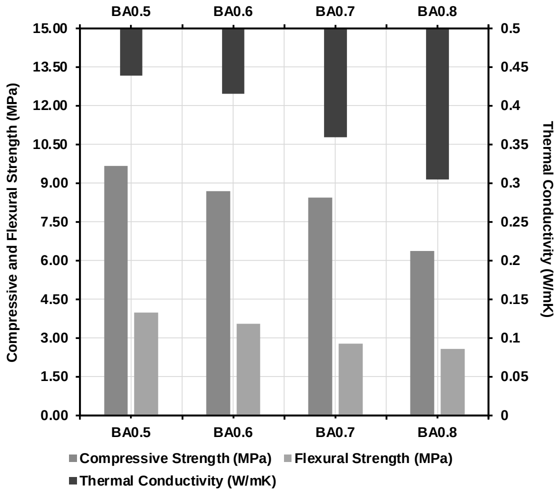

Figure 3. Mechanical and thermal analysis results

Based on the results of tests conducted on mechanical, fresh state properties, and thermal performance, the BA0.7 mixture was selected for 3D printing experiments. This choice was motivated by its ability to provide sufficient thermal insulation while maintaining reasonable strength without significant compromise. Consequently, 3D printing tests were successfully carried out using this mixture (Figure 4), showcasing its balance between structural integrity and thermal efficiency. These experiments validated the material's potential for diverse construction applications, highlighting its adaptability and effectiveness in addressing both environmental and functional demands.

4. Numerical Analysis of 3D Printed Wall Structures

After determining the necessary properties of the developed material, numerical modeling studies were initiated to analyze the energy performance of 3D printed wall models with different designs and configurations. To validate the energy performance results of the designed wall configurations, the wall configuration found in the study conducted by Alkhalidi and Hatuqay (2020) was initially modeled. Thermal transmittance (U) results were compared to ensure model validation. Numerical models for insulated wall configurations with voids were developed using the Abaqus finite element software.

For validation, the wall configuration modeled in the study by Alkhalidi and Hatuqay (2020) was integrated into the developed numerical model (Figure 5). External and internal temperature boundary conditions were set at 0 and 40 degrees, respectively, as in the study. Convective surface heat transfer coefficients for external and internal environments, according to EN ISO 6946, were determined to be 25 W/(m²·K) and 7.69 W/(m²·K), respectively. To introduce these boundary conditions, two surface film conditions were defined to enable interaction between surfaces. The model used heat transfer elements (DC3D8 elements) for meshing. Dimensionally and structurally, the model adhered faithfully to the work of Alkhalidi and Hatuqay (2020). For the sample modeled with voids, mesh sizes were set to 10mm globally and 2mm for the edges, as in the study. The numerical model was conducted through a steady-state heat transfer analysis. The obtained U value from the model was compared with the study results for validation. The U value for the wall modeled with air layer was calculated as 1.64 W/m²K, showed high similarity to the adopted study for model validation (1.87 W/m²K).

Figure 5. Numerical model validation a) Adopted model b,c) developed model's configuration

After ensuring the numerical model's validity through validation, five different 3D printed wall cross-section configurations were determined for analysis using the fundamental properties of the developed material (Figure 6).

Figure 6. Designed wall configuration for numerical modelling

Heat transfer along the 3D wall occurs through three main heat transfer modes: conduction, convection, and radiation. All these heat transfer methods were taken into account in the finite element models. The convective effect was simulated by defining convective film coefficients. According to EN ISO 6946, these coefficients were determined to be 25 W/(m^2·K) for external and 7.69 W/(m^2·K) for internal environments. Radiation heat transfer was simulated by applying a specific emissivity radiation coefficient to the 3D printed wall surfaces. This is because the main source of heat transfer within the cavity is the radiation blocked by the cavity surfaces. These cavity surfaces were selected, and a cavity radiation emissivity of 0.7, in line with studies followed in the literature, was assigned. Tie connections were also assigned to reflect the layer behavior of 3D printed walls for the determined total of 5 configurations in the validation model.

Mesh generation techniques and the selection of appropriate element types are directly related to the accuracy of the numerical model. The 3D printed concrete wall and void insulation were modeled using heat transfer solid elements with 3D 8-node linear brick elements (DC3D8). Additionally, considering the convergence of the results, the mesh size was optimally determined. Since the considered 3DPC wall configurations are non-load-bearing, uncoupled heat transfer analysis was performed. The results obtained for the steady-state condition are as follows (Figure 7).

According to the heat flux (HFL) results, it is clearly observed that an increase in the area of the solid medium facilitating heat transfer reduces HFL values. In this context, the increase in the contact points between the front and back surfaces of the wall is associated with an increase in heat transfer between the two surfaces, and this is seen to control the distribution of HFL.

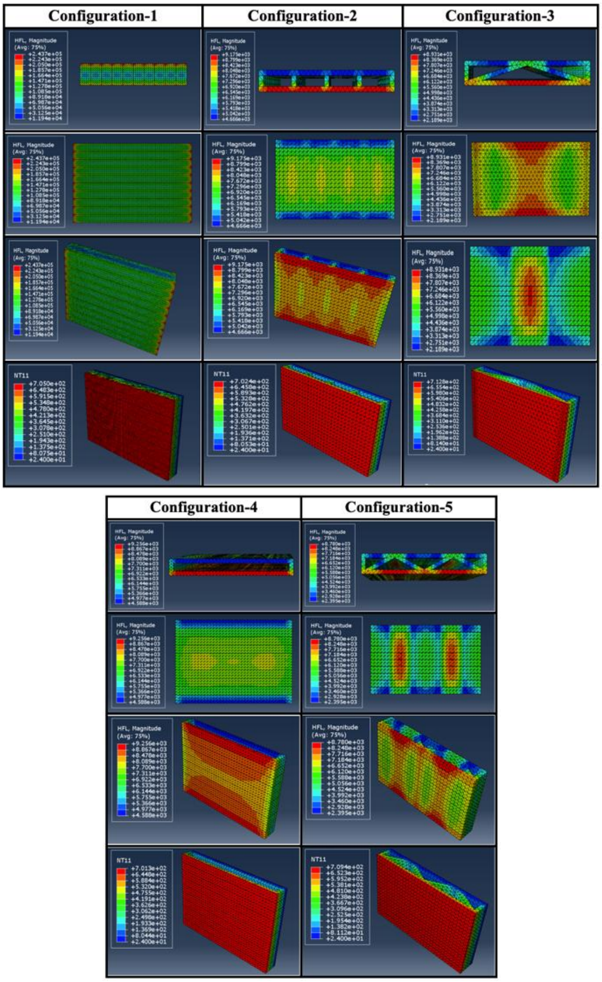

Figure 7. Numerical model results of designed wall configurations

The calculated U-values according to the numerical model results are presented in Table 2. It is evident that an increase in the void ratio has significantly reduced the U-values. However, upon a more detailed comparison, it can be observed that, in the 2nd and 3rd configurations, even though the cavity area decreases slightly, the U-value does not increase. The same situation is applicable to configurations 4 and 5. At this point, the increase in the contact points between the front and back surfaces of the walls in these configurations has control over the U-value. Since the difference in the number of contact points between configurations 2 and 3 is only 1, the U-value does not change significantly. However, in configuration 5 compared to configuration 4, the presence of an additional 3 contact points has significantly increased the U-value.

Table 2. U-values of 3D printed wall configurations

	Cavity Area (cm^2)	**Material Area (cm^2)**	**U-Value (W/m^2K)**
Configuration-1	0.00	30.00	17.54
Configuration-2	23.86	6.14	4.81
Configuration-3	22.03	7.97	4.12
Configuration-4	16.00	14.00	4.19
Configuration-5	17.50	12.50	6.63

5. Conclusion

This study highlights the development and evaluation of 3D-printable, waste-based alkali-activated materials with enhanced thermal insulation properties. By utilizing brick waste as both the binder and aggregate phases, the research addressed critical challenges related to resource scarcity, environmental impact, and energy efficiency in the construction sector.

Experimental results demonstrated that increasing aggregate content improved buildability and thermal insulation while reducing flowability and mechanical strength. The mixtures exhibited a clear trade-off between these properties, necessitating careful selection based on specific construction applications, such as intricate designs or load-bearing structures. Thermal conductivity analyses revealed that higher aggregate content decreased thermal conductivity due to the porous nature of the brick aggregates compared to natural aggregates, enhancing insulation performance.

Numerical simulations of 3D-printed wall configurations provided insights into the thermal behavior of the designed materials. The analysis validated the energy performance of different wall designs, demonstrating a significant reduction in thermal transmittance (U-values) with increased void ratios. However, the configuration and distribution of contact points between wall surfaces also played a critical role in determining U-values, highlighting the complexity of optimizing thermal performance through wall geometry.

Overall, this study underscores the potential of integrating recycled materials into 3D printing processes to achieve sustainable construction practices. The developed alkali-activated materials offer a promising pathway for reducing greenhouse gas emissions, improving energy efficiency, and supporting the transition to a circular economy. Further research is recommended to refine material formulations, optimize wall designs, and expand the applicability of 3D-printed solutions across diverse construction scenarios.

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