

NUMERICAL MODELING OF BURIED SEGMENTED PIPELINE RESPONSE
TO EARTHQUAKE GROUND MOTION

by

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“Now I will have fewer distractions.”

— Leonhard Euler’s remark on the loss of vision in his right eye.

ABSTRACT

NUMERICAL MODELING OF BURIED SEGMENTED PIPELINE RESPONSE TO EARTHQUAKE GROUND MOTION

Earthquakes severely impact infrastructure, disrupting essential services like water pipelines. Understanding and improving the mechanical behavior of pipelines and their joints under permanent ground deformations is crucial to minimize potential damage and ensure the continuity of their service. In this study, three-dimensional numerical models of buried segmented ductile iron pipes were developed to investigate their nonlinear behavior under lateral strike-slip surface faulting. Fault crossing angle, position, and vector were selected as variable parameters, and pipeline performance was evaluated based on different faulting scenarios and two soil models covering a total of 20 cases. The numerical models were validated using experimental and finite element data from the literature. The results indicate that when the fault crossing angle was perpendicular to the pipeline axis, the system was able to accommodate the most amount of fault displacement. Narrower fault crossing angles induced axial separations or compression to the pipeline. Under compression, the pipeline undergoes joint crushing failure by exceeding the material limits, while tensile forces primarily lead to joint pullout. A joint interlocking phenomenon was observed in tension cases when the fault crossed directly through the joint and effectively delayed or prevented failure by uniting adjacent pipe segments. The results also highlighted the sensitivity of pipeline performance to design criteria, where a 2 cm increase in pullout threshold resulted in up to 20 cm of additional fault displacement capacity. Lastly, the study emphasized that while soil models influenced pipeline response under tension, their effect was limited in compression scenarios due to more premature failure.

ÖZET

GÖMÜLÜ EKLEMELİ BORU HATLARININ DEPREM YER HAREKETİ ALTINDA DAVRANIŞININ SAYISAL MODELLEMESİ

Deprem altyapılara önemli hasarlar vermekte ve hayatın devamlılığı için önemli servisler olan su hatları kullanım dışı kalabilmektedir. Boru hatlarının kalıcı zemin deformasyonları altındaki mekanik davranışının anlaşılması ve iyileştirilmesi, olası hasarı azaltmak ve servis sürekliliğini sağlamak açısından büyük önem arz etmektedir. Bu çalışmada, gömülü eklemli sünek demir boruların üç boyutlu numerik modelleri kurulmuş, doğrultu atımlı yüzeyel faylanma altındaki doğrusal olmayan davranışı incelenmiştir. Boru hattı davranışında kritik rolü olan fay geçiş açısı, pozisyonu ve vektörü değişken parametreler olarak seçilmiş, farklı faylanma senaryoları ve zemin modelleri altında performansı toplamda 20 senaryo ile değerlendirilmiştir. Modellerin geçerliliği literatürdeki laboratuvar deneyleri ve numerik analizler kullanılarak doğrulanmıştır. Sonuçlar, fay hattının boru hattı eksenine dik olarak geçtiği durumlarda sistemin en fazla deplasmanı tolere edebildiğini göstermektedir. Daha dar fay geçiş açıları boru hattında eksenel ayrılmalara veya basınca neden olmuştur. Boru hattında, basınca maruz kaldığında malzeme sınırlarını aşarak eklem ezilmesi; çekme kuvvetlerinin etkili olduğu senaryolarda ise eklemlerin ayrılması ile sonuçlanan bir hasar gözlemlenmiştir. Çekme senaryolarında fay hattı doğrudan bir eklemde geçtiğinde boruların birbirine kilitlendiği bir olay gözlemlenmiş, ve bunun hasarı geciktirdiği veya tamamen önlediği tespit edilmiştir. Sonuçlarda boru hattı performansının tasarım kriterlerine oldukça duyarlı olduğu görülmüştür; örneğin, eklemlerin ayrılma eşiği kriterindeki 2 cm'lik bir artış, sistemin 20 cm kadar daha fazla fay deplasmanına dayanmasını sağlamıştır. Zemin modellerinin çekme durumlarında borunun davranışına etkili olduğu, basınç senaryolarında ise erken hasar nedeniyle etkisinin sınırlı kaldığı görülmüştür.

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LIST OF SYMBOLS

c	Cohesion of the soil
d	Burial depth to the pipe centerline
e	Pipe wall thickness
E	Modulus of elasticity
L	Pipe segment length
L_{eng}	Joint engagement length
L_{seal}	Gasket sealing length
n	Ramberg–Osgood strain hardening exponent
P	Joint depth
ΔU_{rel}	Relative axial displacement of the joint elements
U_{crit}	Critical expansion limit
α	Ramberg–Osgood material fitting constant
β	Fault crossing angle
γ	Fault crossing position
δ	Fault displacement vector
ε	Strain
μ	Friction coefficient
ν	Poisson's ratio
σ_n	Normal stress acting on the failure plane
σ_y	Yield strength of the material
τ	Shear strength of soil
ϕ	Internal friction angle of soil
ψ	Dilation angle of soil

LIST OF ACRONYMS/ABBREVIATIONS

2D	Two dimensional
3D	Three dimensional
ALLIE	Internal energy
ALLKE	Kinetic energy
C3D8R	8-node linear brick element, reduced integration with hour-glass control
C3D8RH	8-node linear brick element, reduced integration with hour-glass control, hybrid formulation with constant pressure
DE	Outer diameter
DI	Ductile iron
DN	Nominal diameter
ETOTAL	Energy balance
FE	Finite element
FEM	Finite element method
HDPE	High-density polyethylene
ISO	International Organization for Standardization
MS	Medium dense sand
PGA	Peak ground acceleration
PGD	Permanent ground deformation
PGV	Peak ground velocity
PVC	Polyvinyl chloride
SC	Soft clay
TGD	Transient ground deformation

1. INTRODUCTION

1.1. Problem Statement

For millennia, pipelines have been crucial parts of infrastructure that allowed human civilization to sustain itself. The Romans used them for their water supply. They also repurposed these conduits as storm sewers. Some civilizations, like the Mayans, were even using pipelines for pressurized water plumbing systems. Although they are now often associated with the energy industry, they serve a lot more functions than commonly recognized. From ancient aqueducts and irrigation channels to modern-day pipelines and transportation networks, pipeline systems serve as conduits for the transportation of vital resources such as water, oil, gas, and other commodities essential for sustenance, industry, and energy production, thereby enable the flow of resources and trade. For this reason, they are often termed as “lifelines”.

While it is evident that pipelines play an important role in society’s infrastructure, the significance of maintaining pipelines in working conditions after a possible earthquake cannot be overstated. Several reasons why the safety of pipelines must be ensured can be identified, as their destruction could pose a risk to the safety of life, public health, property, economy, as well as the environment. There are numerous examples of devastating results emerged from pipeline failure caused by earthquakes. In short term, failure of pipelines during earthquakes may prevent critical resources such as water from reaching emergency services including firefighting and healthcare, leading to catastrophe. As an example, in the 1906 San Francisco earthquake, it was estimated that 80 percent of the total damage was caused by fire which destroyed total of 12 km² area. And the reason that the fire couldn’t be dealt with was the lack of water due to the failure of the water supply network of the city. On top of this, the city was also deprived of drinking water as well, which led to epidemics. Another example is the 1923 Kanto earthquake, in which 40 percent of the city got destroyed by fire, and rupture of pipelines led to floods in some areas. Firefighting efforts were

obstructed by inadequate water pressure in pipelines and limited supply of water (Ari-man and Muleski, 1981). The 1999 İzmit earthquake in Turkey resulted in severe damage to the natural gas pipeline network, leading to widespread fires. More recently in Kahramanmaraş 2023 earthquake, fault displacement led to explosions along the Kahramanmaraş–Gaziantep natural gas transmission line. Which led to gas supply of Gaziantep and Hatay stop completely. In addition, extreme fault offsets caused failures in water transmission pipelines as well (Uçkan et al., 2024).

Buried pipelines pass through diverse geologic terrains and encounter a variety of soil conditions and seismic hazards. In critical zones where large differential movement is expected to occur in ground, such as near fault zones, soil shear planes or transition zones, severe damage to a pipeline may be induced. Moreover, pipeline deformations can also be induced by local hazards such as liquefaction, faulting, regional subsidence, earthquake induced settlement, lateral spreading etc. Avoiding these zones is ideal, but sometimes pipelines must be routed through them due to factors like land availability or environmental constraints. Because of these reasons, seismic performance of segmented pipe joints needs to be well enough to withstand severe ground deformations and overall site response in case of earthquake activity.

When designing pipelines and facilities above the ground, the governing factor is inertia effects. However, in design cases that include buried pipelines, the pipe is assumed to be moving with the surrounding soil, meaning that the effect of inertia forces due to the pipe are insignificant. According to Singhal, “It is the relative motion of two points on a buried pipeline, due to the differential ground motions, which provides the governing design criteria” (Singhal, 1984). Besides, the two possible ways that an earthquake wave could damage a buried pipe is either by through wave passage (transient ground deformation) or through permanent ground deformation (Moser and Folkman, 2008). Transient ground deformation (TGD), which is an effect of seismic wave propagation, usually affects large areas. Whereas permanent ground deformation (PGD), is a result of locally occurring phenomena mentioned before. Although both mechanisms work distinctly, they can cause major deformations in the ground that

lead to engineering problems while designing pipelines, such that they need to be compensated in some way. The governing factors of performance of segmented pipelines under these large ground deformations are strongly dependent on the axial pullout and compressive load capacity of their joints, as well as by the limits of their ability rotate.

In between those two phenomena, a general consideration is that PGDs pose a greater hazard to segmented pipelines, due to more local and abrupt changes in the soil domain. A pipeline's response to earthquake induced PGD is typically categorized as large deformation cases due to the severe displacements that occur in nearby ground during seismic events. Because of these large deformations, the elastic limit of a pipeline is often exceeded, and the nonlinear behavior gets introduced to the case. This behavior, as well as the behavior and interaction of the surrounding soil needs to be considered while establishing pipeline performance to represent their in-situ behavior accurately. And design seismically more resilient lifeline infrastructures.

It is possible to create highly detailed simulations with FEM (Finite Element Method) that takes every real life aspect into account using advanced computer-aided engineering software. However, simulations that include effects such as PGDs are complex, as the process demands high computational power, time, and skilled analysts. Especially given that segmented pipelines cannot be modeled using shell elements like the continuous pipelines, which are relatively far simpler to simulate due to their nature. Abaqus/CAE 2024 software can be utilized for such engineering problems as it is deemed as a robust tool for complex problem specific FEM models.

In summary, buried pipelines play a critical role in infrastructure. Past incidents have shown that PGDs such as strike-slip surface faulting can lead to severe pipeline service disruptions. For these reasons, the seismic resilience of segmented ductile iron pipelines must be evaluated against various ground deformation hazards.

1.2. Scope of Study

The concern of this thesis is to conduct multiple sophisticated numerical simulations of response of buried segmented pipelines subjected to various types of local deformations in ground caused by different cases of faulting. And develop an understanding of their behavior under change in Fault Crossing Angle (β), Fault Crossing Position (γ), and Fault displacement vector (δ). Two soil models with significant difference in shear strength are defined with Mohr-Coulomb Plasticity model and utilized as an additional parameter. By leveraging state-of-the-art computational tools and methodologies, the study aims to provide robust models which will offer more detailed and accurate results compared to conventional modeling techniques. Thereby contributing to the understanding and development of more resilient pipeline infrastructures.

1.3. Thesis organization

The thesis consists of six main chapters. The first chapter provides a historical background and brief introduction to pipelines, their importance, earthquake performance, types of seismic hazards, as well as ways to establish their response. Lastly, the research scope and organization of thesis are given.

Chapter 2 of the thesis conducts a review of prior research on buried pipelines and their response to earthquake ground motion, while highlighting significant contributions, as well as ongoing methodologies and applications in the literature. The chapter also provides the fundamental principles of segmented pipeline systems.

Chapter 3 presents the preliminary validation model and the conducted numerical analysis. The properties of the model in the selected study as well as how they are implemented to the newly prepared model on Abaqus are discussed. Then, the output results are compared to verify that the numerical models developed in the oncoming sections of the study are rational.

Chapter 4 provides the main study and base models that are conducted for the thesis. The chapter starts by introducing the studied parameters, cases, and the selected tools. Then, a general information about the models is given along with how they are prepared, and which factors they account for. This includes material definitions, contact definitions and meshing approach. The process of benchmark analyses and optimization of the models are also informed to the reader in this part. Next, critical aspects that needed to be considered are mentioned besides how they impact the simulations, and which measures were taken to address them. Lastly, the analysis procedure section explains the series of two-step simulations conducted for the series of studied FEM simulations. The boundary conditions, loading methods, and quasi-static assumptions that are made are outlined in this subchapter. Additionally, the choice of the solver for both steps to avoid the complexities of import analysis is explained.

Chapter 5 provides results of the conducted twenty FEM analyses. General observations are addressed initially. Then, the results of the models are compared with each other using various graphs and tables to determine critical areas of concern and establish pipeline performance under considered parameters.

Chapter 6 outlines the conclusions of the study interpreted from the obtained results. Finally, a discussion on possible fields of application that the assertion outcomes can contribute to are given, along with recommendations for future work of this study to expand.

Finally, the references chapter includes all the scholarly articles, books, technical reports, and other sources cited throughout this thesis. The listed references form the basis for the theoretical background of the different numerical modeling approaches utilized in the study.

2. REVIEW OF THE LITERATURE

In this chapter, a review of prior research on buried segmented pipelines is conducted, while highlighting significant contributions as well as ongoing methodologies and applications. In addition, basic information is provided regarding buried pipeline systems. The purpose of this chapter is to establish a foundation of knowledge about pipelines, their current state of the art, the mechanisms and extent of their damage during earthquakes, and the methods for evaluating their seismic performance. This will assist in gaining a better understanding of the topic, as well as be a reference guide in establishing the numerical model in terms of identifying the critical areas of concern during the modeling phase.

Regarding the previous studies, there are many reviews, case studies, analytical, numerical, and experimental work done to understand the response of pipelines undergoing earthquake motion. And a vast selection of books and codes emerged for designing buried segmented pipelines with the ability to withstand large deformations on joints. For example, "Buried Pipe Design" by Moser and Folkman as well as "Seismic Design of Buried and Offshore Pipelines" by O'Rourke and Liu are widely regarded as fundamental texts for understanding the principles of buried pipe design. They provide a detailed explanation and guidance on the design of buried pipes. Additionally, they cover topics that are important to this thesis such as soil-structure interaction, pipe materials, load, and stress analysis.

In the first part, the types of connections and materials used in buried segmented pipelines are reviewed. The second part covers the seismic damage and vulnerability of buried pipelines by examining the types of damage induced by earthquakes, their failure modes, as well as damage estimation methods in the literature. The third part concerns various methodologies used to evaluate the seismic performance of buried pipelines. Moreover, the section explores the analytical, numerical, and experimental methods as well as the studies in literature to understand the pipeline behavior under

earthquake ground motion.

2.1. State of the Art in Buried Segmented Pipelines

Buried segmented pipelines are composed of two elements, namely the pipe segments and the joints. To conduct a numerical analysis, their material, axial, and flexural behavior must be studied first. In this part, connection types are explored first while reviewing their commonly used models, benefits, limitations, and applications. The effectiveness of each type of connection in maintaining pipeline operational after seismic events is discussed. Next, the part covers the range of materials employed in pipeline construction, including steel, cast iron, ductile iron (DI), concrete, and newly emerging high-density polyethylene (HDPE) as well as composite. The review includes how material choice impacts the overall performance and longevity of the pipeline, especially in seismic zones where flexibility and resilience are crucial.

2.1.1. Segmented Pipelines

According to Banushi et al. (2020), pipeline connections are divided into three types by their boundary conditions. Unrestrained, fully restrained, and hybrid systems (Banushi et al., 2020). However, a more common way to differentiate pipelines is to simply consider them as either segmented or continuous. Segmented pipelines, being the subject of this dissertation, are considered as pipelines with much lower axial and rotational stiffness at their joints compared to the rest of their body. Because of their low stiffness at joints, segmented pipelines subject to axial tension will expand and pull apart first before the material can leave the elastic region and fail (O'Rourke and Liu, 2012). Banushi et al. (2020) state that traditional segmented pipelines have less resistance to the effects of PGD compared to continuous systems because of the same reason. To overcome this issue, hybrid system pipelines are being developed and used in practice. In case of severe motion, hybrid segmented joints displace axially before locking up and acting as a continuous system. The difference between hybrid systems and traditional unrestrained joints is that, after the locking mechanism engages, these

systems use anti-pullout mechanisms that don't allow the continuous behavior to be disrupted. A common application for these systems is ductile iron pipes, which are still being used throughout the industry for mainly water networks (Banushi et al., 2020).

2.1.2. Flexible Joints

Flexible joints are essential in the design of segmented pipelines that are prone to earthquake ground motion. Several researchers, including Isenberg and Richardson (1989), Ballantyne (1992), and Wang (1994), have proposed the use of flexible joints as a mitigation strategy for pipelines exposed to permanent ground displacement. The main working principle of modern flexible joints varies and normally involves hyperelastic gaskets to provide a seal between two adjoining pipe segments. The joint variations can be either unrestrained or have a locking mechanism to prevent axial pullout. The variations include mechanical joint, bell-spigot joint, bellow joint, flange-lock joint, telescoping sleeve, expansion joint, ball joint, etc. The most common method to connect segmented pipelines is the push-on joints with a bell and a spigot, as it is widely utilized in real life such as in water networks (O'Rourke and Liu, 2012).

Valsamis et al. (2020) explored the efficiency of segmented pipeline joints. They stated that traditional methods such as increasing pipe wall thickness or reducing soil-pipe friction for continuous pipelines are considered effective only for small ground deformations. However, for larger deformations, the method is cost-prohibitive. Because of this reason, they further explored the performance of flexible joints. The parametric study included a series of three dimensional (3D) nonlinear finite element analyses to simulate PGD resulting from strike-slip and normal fault rupture on conventional flexible joints. The comparison included many parameters. For segmented pipelines specifically, the joint-to-joint distances of 6 m, 8 m, and 12 m are compared. The results showed that flexible joints are an efficient way to minimize the effects of strike-slip faults with fault crossing angle (β) of 90° and large magnitudes of displacements, as well as the effects of perpendicular crossing of normal faults. They also state that bellow joints offer the best performance due to their ability to excellently accom-

moderate longitudinal and bending loads. Their results finally show that these joints are particularly beneficial in fault displacements exceeding 3.3 times the pipeline diameter, where other mitigation strategies fail.

2.1.3. Pipeline Materials

A variety of materials are being used in the pipeline industry, and no particular one has replaced the others yet. The reason behind this is that each material has its advantages and disadvantages. Material traits such as strength, stiffness, corrosion resistance, weight, flexibility, and ease of installation are considered when choosing for construction of a pipe. The material which the pipe will be constructed from plays a significant role in its earthquake performance. For example, a material with high flexibility could help ensure greater strains, while sacrificing from some other aspect. Or, a material such as cast iron may have extra strength but could be highly susceptible to corrosion. Selecting a material that combines strength and durability with the ability to undergo expected deformation can mitigate the risk of failure and ensure the pipeline stays functional. In addition, the pipe material and the joint design need to be optimized together for the seismic resilience of a pipeline system. And the pipeline must be durable enough to last until the end of its designed lifespan.

There are generally two classes of piping materials as rigid or flexible. For a pipe to be considered flexible, it must have the capacity to deflect 2 percent or higher without compromising its structural integrity. Otherwise, it is considered to be rigid (Moser and Folkman, 2008). To give an example, cast iron and concrete pipes are classified as rigid, whilst ductile iron, steel, or thermoplastic pipes such as HDPE and polyvinyl chloride (PVC) pipes are considered flexible. For each material class, there are critical limitations which must be evaluated. Pipes that are made of rigid materials need to be able to resist wall stresses, and a pipe made from a flexible material must be stiff enough to prevent failures such as buckling. The prevailing pipe materials around the globe are cast iron, ductile iron, steel, concrete, asbestos cement, composites, as well as thermoplastics such as HDPE and PVC.

O'Rourke et al. (2012) conducted an analysis of the seismic performance of underground pipeline systems by observing data from the Christchurch 2010 and 2011 events. Their analysis included impacts of both permanent and transient ground deformation. After exploring 1645 cases of pipeline repairs, they stated that repair rates for pipelines that are made of brittle materials (e.g., asbestos cement or cast iron) were significantly higher in areas affected by PGD compared to more ductile pipelines such as polyvinyl chloride (PVC) and medium-density polyethylene. Moreover, they highlighted the seismic resilience of polyethylene pipelines in the gas distribution system as they sustained minimal damage despite being exposed to severe ground deformations. Lastly, they suggested the importance of material ductility as well as the need for improved joint designs in the performance of pipeline systems.

2.2. Earthquake Damage in Pipelines

There are two possible ways that an earthquake motion could damage a buried pipe. Either through wave passage (transient ground deformation) or through permanent ground deformation. Ariman and Muleski (1981) are among the first to partially make this distinction. They stated that a pipe can be either damaged by the traveling seismic wave or the relative movement of soil surrounding the pipe. However, they excluded the effects of fault rupture and landslides. O'Rourke and Liu state that permanent ground deformations (PGD) are caused by movements of the surrounding soil such as those resulting from surface faulting, lateral spreading due to liquefaction, and landslides. According to Nair et al. (2018), faulting causes the most extreme damage compared to others. Although hazards caused by PGD are often limited to small regions within the pipeline network, their potential for damage is very high because they impose large deformations. Contrary to this, hazards resulting from transient ground deformation typically occur over wide geographical areas with lesser damage rates (e.g., lower pipe breaks and leaks per unit length of pipe). They point out the difference between these two cases by giving the example of the 1906 San Francisco earthquake. According to T. O'Rourke et al. (1985), only 5% of the zones affected by the earthquake at which lateral spreading has occurred are responsible for as much as

52% of all pipeline failures. Whilst the remaining 48% is attributed to wave propagation. This means that the damage rate in the small, isolated areas subject to PGD was significantly higher than that due to wave propagation. This effect, according to O'Rourke and Liu (2012), can be again observed in many events such as the 1995 Kobe, 1999 Kocaeli, and 1999 Chi-Chi earthquakes.

Liu et al. (2020) conducted a systematic and parametric study of segmented pipeline connections as well as their influence on joint deformations. The parameters they considered are the shapes of the connections, ground conditions, pipe diameters, branch angles, seismic incident angles, and the properties of the incident wave. Moreover, they considered five connection shapes along with five other parameters (soil properties, pipe diameter, branch angle, incident wave angle, and input ground motion) to analyze their influence. They stated that the five types of pipe connection can positively influence the joint deformations of segmented pipelines by up to 50%. Another significant finding they proposed is that, for cases of transient ground deformation, joint deformations are more prone to be influenced by characteristics of the pipe (specifically connection shape and pipe diameter) and soil conditions rather than the wave properties. Finally, they proposed an influence coefficient that quantifies at which degree a pipe connection may influence the joint deformation and suggest that 0.65 is enough to ensure the joint stays operational.

2.2.1. Permanent Ground Deformation

O'Rourke and Liu (2012) states that the seismic behavior of a buried segmented pipeline undergoing PGD generally depends on the type of ground deformation (e.g., longitudinal or transverse), the amount of ground movement, the extent of the deformation zone, as well as the pattern of ground movement. According to Banushi et al. (2020), longitudinal PGD has a higher chance of damaging a non-seismically designed buried pipeline compared to transverse PGD. They associate the reason for this due to lower flexibility of the pipeline in the axial direction. Regarding the type of deformation, Suzuki (1988) stated that for segmented pipelines, damage caused by longitudinal

PGD was more commonly encountered than that due to transverse PGD in the 1964 Niigata earthquake. O'Rourke and Liu (2012) accounted the reason for this difference to the pullout of joints during the tension of the ground in numerous examples. They also state that, for the pattern of the ground movement, the uniformity or the rate of change of the deformation in the ground strongly influences the distribution of strain along the pipeline.

2.2.2. Transient Ground Deformation

During strong ground motion, the effects of wave propagation on the pipeline are inevitable. TGD refers to the effects of seismic waves traversing the ground surface, which results in strains and curvatures in the body of soil. Ground strain induced by wave propagation is given by the equation

$$\varepsilon = \frac{V_{max}}{C}$$

where V_{max} is the peak horizontal particle velocity; and C is the effective propagation velocity of seismic waves along the ground surface (O'Rourke and Ayala, 1993).

Pipeline damage due to wave propagation effects is usually correlated with the peak ground motion parameters, as well as the wave properties. The peak ground motion parameters for this matter are peak ground acceleration (PGA) and peak ground velocity (PGV). However, O'Rourke and Liu (2012) state that PGV has a greater effect because it is more directly linked to the strain and stress induced on the ground and eventually the pipeline. On the other hand, PGA is more often indirectly associated with PGD hazards such as landslides and others. And lastly, key wave properties in TGD include propagation velocity and wavelength. The subsections below review these seismic parameters as well as how they are attenuated during strong ground motion.

2.2.2.1. Wave Propagation Properties. The primary type of wave that induces the most strain on the near-surface ground is the surface waves as substantial damage to

pipelines was observed due to them over the years. This is attributed to the fact that surface waves can generate larger ground strains since they have lower phase velocity and frequency, as well as they are higher in amplitude. Besides, during their travel through the ground surface, surface waves exhibit motions that cause both axial and bending strains on buried pipelines throughout pipe-soil interaction. Moreover, among the types of surface waves, particularly Rayleigh waves are generally more destructive and produce greater ground strain compared to Love waves. As for body waves, S waves tend to cause larger ground strains such that they carry more energy than P waves.

Bi and Hao (2012) conducted a study to investigate the combined effects of spatial variations in ground motion and local soil conditions on buried segmented pipelines, including wave incident angle and joint stiffness. They considered a case of heterogeneous soil deposits on top of base rock surrounding the pipeline. And calculated the site amplification effect by using a one-dimensional wave propagation theory. To do this, they assumed that the motions of base rock are out-of-plane SH waves or combined in-plane P and SV waves. And stochastically formulated the differential axial and lateral displacements between segments in the frequency domain. In conclusion, they observed from their numerical results that the influence of the mentioned parameters is significant in terms of resulting differential displacements on segments. Moreover, the significance is greater if the soil properties surrounding the two segments are the same. And the effect of an increase in approaching wave incident angle results in greater relative displacements in segments, and eventually, greater damage. As for joint stiffness, they stated that as the joint gets stiffer, the relative difference between the response of segments becomes smaller. Their final suggestion is that damage due to transient ground deformation can be mitigated by designing the joints stiffer.

2.2.2.2. Ground Motion Parameters. As mentioned before, the significant ground motion parameters for buried pipelines are PGV and after that PGA. They are characterized by attenuation relationships, which are empirically derived mathematical expressions that relate various seismic parameters such as magnitude, distance, and local

geological properties of the site to the resulting strong ground motion parameter.

There are vast number of attenuation relations for various seismic parameters. Particularly useful for the case of pipelines, Campbell and Bozorgnia (2008) presented an attenuation model that relates the PGV and PGA as a function of the closest distance to the rupture for moment magnitudes (M_w) between M_w : 4 and M_w : 8.5 and epicentral distance of up to 200 km. Their model is for 5% damped linear elastic response spectra for periods between 0.01 and 10 seconds, and utilizes a set of recordings. Specifically for the case of strike-slip faulting and average shear wave velocity (V_{S30}) of 760 m/sec, they pointed out the parallel correlation between PGV and earthquake magnitude. Such that PGV values are decreasing as the magnitude of the earthquake decreases and vice versa. For the same conditions, the influence of correlation decreases with increasing distance. This implies that larger earthquakes result in higher velocities for buried pipelines as well as increasing distance from the source decreases the amount of expected local PGV on a given pipeline (O'Rourke and Liu, 2012).

2.2.3. Modes of Failure in Segmented Pipelines

The failure modes that could occur in a buried pipeline due to earthquake motion mainly depend on the type of pipeline connection. Specifically, O'Rourke and Liu (1999) categorized the most common damage mechanisms observed on segmented pipelines as failure due to axial pullout, crushing of bell or spigot, and circumferential damage. Nair et al. (2018) state that the most vulnerable part of segmented pipelines during earthquakes is the joints, and their most encountered failure mode is the axial pullout failure. However, the information about failure thresholds in segmented pipelines is not as available as in continuous systems due to the complex failure criteria of segmented pipelines.

Singhal (1984) investigated the axial, bending, and torsional responses of ductile iron pipes by exploring the effects of burial depths. The study introduced a joint stiffness matrix that accounts for all three failure modes to predict the seismic response

of buried segmented pipelines. In conclusion, the study showed that the mechanical properties of the gasket as well as the geometry of the joint govern the axial, bending, and torsional resistance of the joint. As for burial depth and frictional properties of soil, the study shows that there is a relation between the response of pipelines. Singhal also identified the importance of joint geometry in preventing excessive stresses during bending and axial compression and suggested that minor modifications in design such as increasing the joint depth and bell socket diameter could enhance the seismic performance of pipelines. These suggestions can be considered as the foundation of modern segmented pipeline designs.

2.2.3.1. Axial Pullout Failure. Axial pullout failure occurs within the vicinity of tensile ground strain. It is caused by unrestrained axial behavior of joints enabling segments to move and pull out from each other during severe motion. Moreover, the shear strength of the joint gaskets is significantly less than the tensile strength of the body of the pipe. This indicates that joints are the critical zones of segmented pipes, which is the main reason behind pullout failure.



Figure 2.1. Replacement and repair of an axial pullout failure observed in 1999 Kocaeli earthquake (adapted from Ansal et al., 2008).

In their research, El Hmadi and O'Rourke (1990) focused on predicting the effects of seismic wave propagation and permanent ground deformation (PGD) by using analytical procedures. Their model for evaluating the response incorporates the non-linearity resulting from the change in characteristics of the soil and joints of the pipeline. By using their method, the probability of exceedance is obtained for multiple levels of relative joint displacement as a function of ground strain. They used Winkler soil springs in their analytical model. They assumed the springs as continuous in the formulation and modeled each pipe segment using truss and beam elements. Also, joints between each segment are represented by nonlinear axial and rotational springs. Another aspect of this model is that it does not consider longitudinal cracking as well as hoop stresses which induce a flattening effect on the pipe. To be able to create this analytical model, the required information are properties of the pipe material, outside diameter, thickness of the wall, and length of the segment, which are all assumed to be deterministic. Monte Carlo simulations are used to generate a probabilistic distribution of joint displacements, and their results show the likelihood of leakage at varying levels of ground strain. Their research resulted in several points. They concluded that joint pull-out is a significant failure mode for segmented pipelines, especially for cast iron pipes with lead caulked joints. A critical finding of their study is that joint pull-out damage ratio is a decreasing function of pipe diameter. Also, the probability of significant leakage is a function of the relative depth of the bell and spigot, meaning that further they are located relative to each other, the chance of significant leakage increases. Another point they made is that the joint pull-out damage ratio is a non-linear function of ground strain. Lastly, they stated that the variability of displacements of joints is influenced by the stiffness of the joint, rather than the properties of surrounding soil.

2.2.3.2. Crushing of Bell or Spigot. Opposite to axial pullout, the crushing of bell or spigot occurs in areas where the compressive motion is induced on the pipeline. In their damage mechanisms for segmented pipelines, O'Rourke and Ballantyne (1992) expressed it as the compressive failure of joints. This failure mode is mostly observed on concrete pipes due to their brittle nature. However, it could also appear in telescoping

form in other more ductile materials such as DI in rare cases. Failure due to the crushing of the bell or spigot appears with circumferential cracks at the end of the pipe segments. According to the observations of Bouabid and O'Rourke's (1993) laboratory axial compression tests, this type of failure on reinforced concrete pipes could start in either the inner or the outer concrete lining. O'Rourke and Liu (2012) also add that the next critical section of the pipe after these cracks becomes the welded steel interface between the segments. Of which the eccentricity causes denting or rarely local buckling that leads to failure. For that reason, both Bouabid and O'Rourke (1994) suggested that for the crushing failure of a bell or spigot of segmented concrete pipelines, the failure criterion should be based on the ultimate compressive capacity of the concrete multiplied by the area of the core only (O'Rourke and Liu, 2012).

Regarding the 1985 Michoacan earthquake, Ayala and O'Rourke 1989 released a technical report that explored the event's effect on water distribution systems in Mexico City. They observed that the damage done to the asbestos cement pipes predominantly occurred in soft soil areas (e.g., near lakes) and specifically at joints of segmented pipelines. Moreover, they concluded that most of the failures observed in the city resulted from the crushing of the joints accompanied by pullout in some instances due to local soil conditions. They also stated that damage at pipe joints was due to insufficient tensional, compressional, and rotational strength.

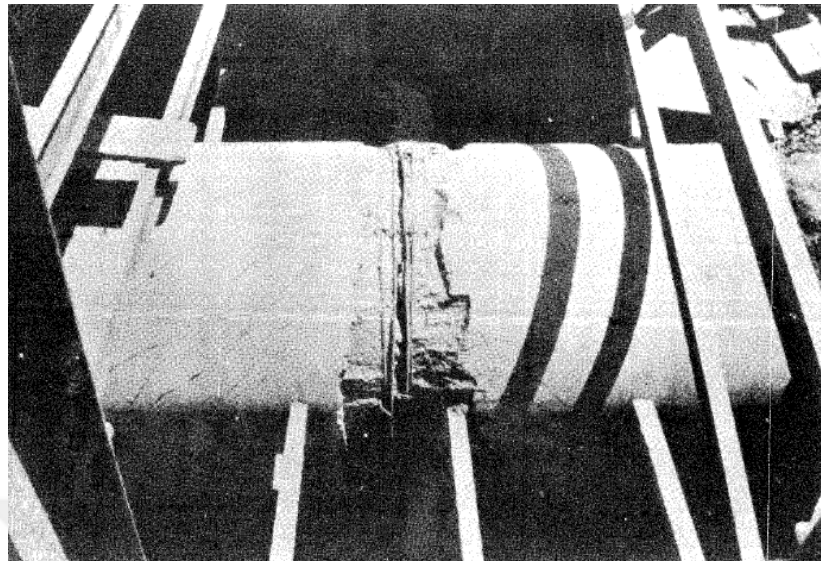


Figure 2.2. Joint crushing failure of a fully restrained “lock jointed” segmented pipe, 1985 Michoacan Earthquake (adapted from Ayala and O’Rourke, 1989).

2.2.3.3. Circumferential Flexure Failure. Circumferential flexure failure is a rare failure mode that occurs where ground curvature subjects the segmented pipeline to bending stresses. It is triggered when the surrounding soil exerts pressure that is significantly higher at the angle of the hoop relative to other angles and is usually encountered with smaller diameter pipes. The bending on the pipeline is accommodated by both joint rotation and flexure of the pipe’s body. Their relative contribution depends on the ability of the joint to rotate and the stiffness of the segment. For flexible segmented pipeline systems, such as those with rubber or hyperelastic gaskets, it is assumed that bending stresses in the body of the pipe start to gain significance only after the joint rotation capacity reaches its limit. This limit changes between different segment design methods and usually is between 4° and 15° . For rigidly connected segmented pipelines, however, the bending stresses are accommodated by the body and the joint when the ground curvature starts. Circumferential failure resulting from bending stresses appears as either flexure cracks on the body or the on joint elements depending on the nature of the material and the joint design. For example, during hoop failure, cast iron or asbestos cement pipes usually crack on the body, while concrete pipes subject

to ground curvature often crack at the bell and spigot ends because of the joint ring eccentricity (O'Rourke and Liu, 2012). Since the failure of the pipe due to bending is governed by both the tensile and compressive strength of the material, El Hmadi and O'Rourke (1989) proposed that, for round and flexure cracks forming due to bending of the pipe, the failure criterion can be based on the ultimate tensile and compressive strains of the material used.

2.2.4. Countermeasures for Mitigation of Seismic Damage

The seismic resilience of pipelines has been explored and improved since the mid-20th century when cast iron pipes were in use. Researchers have come up with many methods for mitigation of seismic damage on buried pipelines. The application of these techniques is essential to minimize the potential of pipeline failure. They are either done to prevent hazards from reaching or affecting the pipe, as well as to strengthen the pipe to resist them. The subsections below describe those which are widely used in buried segmented pipelines.

2.2.4.1. Routing and Relocation. One of the most common and easy methods of mitigation against potential seismic damage in all kinds of engineering structures is to simply avoid the site completely if possible. The method is often preferred since it is considered during the initial design phase, although it is possible to relocate an existing pipeline to a different location. The method is considered more effective for local problems in soil such as those arising from PGD or fault crossings. However, it is considered to be less, if not entirely ineffective against TGD because it affects larger areas.

2.2.4.2. Using Flexible Materials. It is widely accepted that using flexible materials tends to reduce the risk of failure in buried pipelines. In his study, Eguchi (1991) established a fragility relation between pipeline materials and repair rates for various types of ground deformation using the 1971 San Fernando earthquake data. At a given

Modified Mercalli Intensity level, the relation offers higher repairs for more brittle pipe materials such as cast iron and concrete, while less failures for more ductile ones (e.g., DI, HDPE).

2.2.4.3. Limiting the Impact of Soil-structure Interaction. Pipeline placement conditions could be adjusted to avoid site-specific hazards. For example, pipelines can be placed to have a small space around them to accommodate movement near the zones of PGD and strike-slip faults. They can also be placed above the ground with supports if the expected PGD from nearby fault crossing is severe. However, the design needs to be considered carefully because exposing the pipeline introduces the effect of inertia as a result of ground acceleration. They can also be isolated from the ground using different methods to minimize the friction between soil and pipeline. Demirci (2019) listed the available techniques to isolate the pipelines from soil as well as to reduce the effect of soil stiffness. and their basic principles in Table 2.1.

Table 2.1. Available techniques for limiting the impact of soil-structure interaction
(adapted from Demirci, 2019).

Mitigation Technique	Method Type	Benefits
Geotextile	Friction Reducing	Geotextiles reduce the friction at the interface. The anchorage length increases and axial strains are distributed more evenly.
Pumice	Trench Backfill	Loose granular characteristic of pumice exhibit lower stiffness and decreases the friction at the interface. The anchorage length increases and axial strains are distributed more evenly.
Trench Enlargement	Trench Backfill	Allows deformations to be distributed over a greater length and helps prevent the concentration of strain in a localized area.
Geocells and Geogrids	Friction Reducing	Geocells and geogrids reduce the interface friction and minimize the development of strain along the pipeline.
Geofoam	Trench Backfill	Geofoams have a very light unit weight which severely decrease the horizontal and vertical forces applied on the pipeline.
Culvert	No Backfill	There are no soil-pipe friction forces due to absence of interface

2.2.4.4. Using High-Strength Pipes. The strength of pipes could be increased using three methods: wrapping the pipe, increasing the wall thickness, and increasing the material strength. Wrapping of the pipes is usually done with composite wraps that provide additional strength to accommodate part of the axial strain on the pipeline. Meanwhile, increasing both the wall thickness and material strength directly increases the axial and bending strength of the pipeline.

2.2.4.5. Anchoring Effect and Joint Springs. Wham et al. (2019) state that the enlarged portions of the segments such as bells or joint harnesses significantly influence the total axial forces acting on the pipeline. Specifically, as the relative displacement between the soil-pipeline interface increases, these portions act as anchor-like components which progressively contribute to the friction, passive bearing pressure, as well as soil yielding. It should be noted that according to Banushi et al. (2020), due to the requirement of complex analytical solution and validation for each case, the anchoring effect is not considered in current seismic design codes.

During ground movement, the soil-pipeline interface is subject to axial friction forces over the length of the longitudinal PGD zone. Not to be confused with the effects of abrupt longitudinal PGD on pipelines, these forces result in axial tension at the head and axial compression at the toe of the longitudinal PGD zone as well. O'Rourke and Liu (2012) suggest that, in a scenario where all soil properties throughout the PGD zone are the same, the tensile and compressive strains at both the head and the toe should be the same too. However, due to the allowable tensional strain being significantly higher than the allowable compressional strain, they suggest redistributing the soil friction force to make the head part accommodate the majority. For that matter, they offer to place a fixed anchor point immediately before the head of the PGD zone as an option. By doing this, axial forces within the pipeline are redistributed efficiently. Using the same principle, they also offer to place soft springs after the toe of the PGD zone to reduce the compression forces further and mitigate the risk of exceeding allowable strain limits.

2.3. Evaluation of Seismic Response of Buried Segmented Pipelines

2.3.1. Analytical Approaches

The response of buried pipelines can be calculated using analytical approaches. The advantage of analytical methods is that they enable relatively faster calculations. They also don't require much manpower, computational power, or time. Understand-

ing the analytical methods is crucial to the evaluation of the seismic performance of buried segmented pipelines as their use is crucial during the earlier stages of design. The analytical approach offers several advantages over laboratory experiments as well as numerical modeling. Unlike experimental techniques, it does not demand physical setups or specialized equipment. Additionally, it is computationally efficient and requires minimal resources while still estimating results with reasonable accuracy using simple matrices. Analytical methods also allow one to have an idea of the basic principles governing the interaction between pipes and soil as well as the underlying physical mechanism of overall pipeline response. Furthermore, its simplicity makes it accessible for preliminary assessments without extensive computational overhead. Huo et al. (2023) conducted a systematic literature review to investigate the existing analytical methods for pipeline-soil interface problems in cross-sectional, transverse, and longitudinal directions. The summary showed concentration in certain areas of study for the three directions. For cross-sectional analyses, most of the studies are found to be conducted on ovalization. For bending or longitudinal loads, the area of concern is found to be mostly PGD-related issues such as fault crossing or ground settlement. Huo et al. (2023) also observed that the most common method utilized for analytical approaches is the simplistic beam on elastic foundation method. However, due to the complex behavior of segmented pipelines subjected to ground deformation, studies specifically focusing on analytical approaches for segmented pipelines are sparse. A comprehensive study found in this matter is the approach proposed by Wham and Davis (2019).

Wham and Davis (2019) proposed a closed-form analytical solution to evaluate the buried pipeline response to permanent ground deformation by estimating induced axial strains and connection force capacity. The method works for both continuous and segmented pipeline systems and any defined material in the elastic range, and outputs results using a single set of equations for all parameters. It evaluates the relative soil displacement between the pipeline-soil interface for axial block ground deformation pattern, which is an idealization of the relatively uniform longitudinal distribution of PGD among the effected zone. The solution applies this PGD pattern parallel to the pipe axis. Moreover, the model assumes that PGD is only in the longitudinal direction

and does not accommodate for any perpendicular movement of the pipeline. They defined the problem using a set of three differential equations, which outputs the result as pipe strain, pipe displacements, and ultimately the required joint capacity necessary to accommodate the associated level of ground movement.

2.3.2. Numerical Models

The complexity of pipeline-soil interactions necessitates the use of sophisticated numerical models that can account for all relevant factors, including geometrical and material non-linearity, cross-sectional deformations, and advanced soil properties. For segmented pipelines, numerical analyses are essential because they allow for the detailed mathematical expression of these complex physical interactions which analytical approaches are unable to address. Currently, there are two main types of models acknowledged to implement in numerical method: simplistic beam or shell type model which utilizes Winkler soil approach, and three dimensional (3D) solid continuum model. The following subsection provides the variety of developed studies for both models available in the literature in their chronological order.

2.3.2.1. Beam or Shell on Winkler Foundation Model. These models, being the simpler and quicker option, utilize the Winkler soil approach. Introduced by Winkler (1867), the approach considers the surrounding soil medium as a system of finite, discrete, linear elastic springs in all directions. The pipeline itself is meshed using finite elements in either beam or shell (or both) form and is capable of modeling axial, shear, and bending deformations as well as stress and strain data. The disadvantage of this model is that the majority of the soil and site properties (e.g., native properties that amplify earthquake waves, trench dimensions, pipe-soil interface) cannot be directly incorporated into the analyses. However, the model is a broadly accepted method in both literature and practice for establishing pipeline safety at active fault crossings. It is widely recognized by renowned codes and standards around the globe such as Eurocode, ALA, and ASCE as a reliable and streamlined numerical approach.

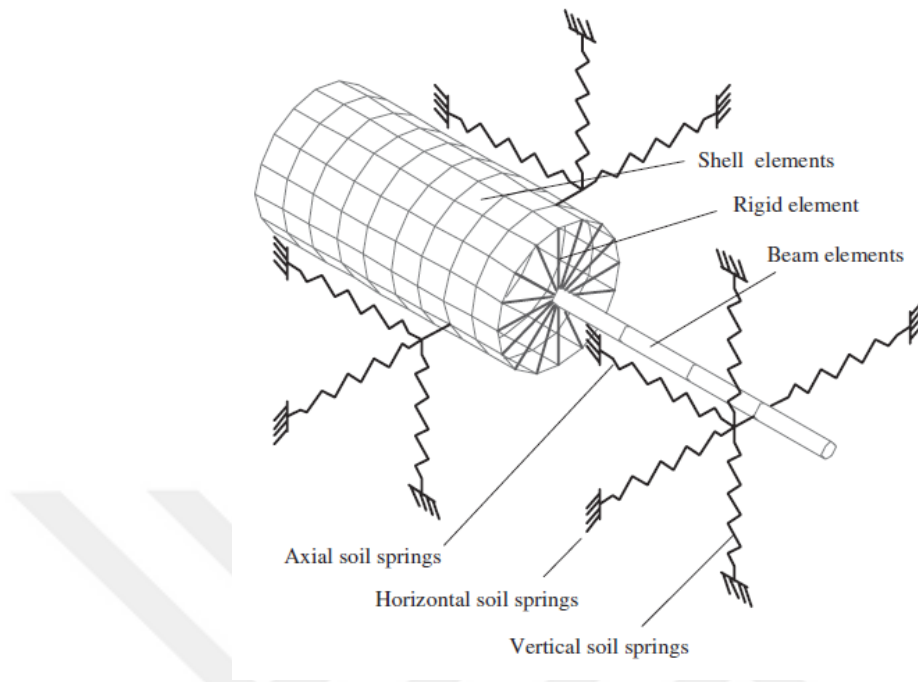


Figure 2.3. Visual demonstration of both simplistic beam and shell models, as well as Winkler soil springs on a hybrid model (adapted from Karamitros et al., 2007).

The beam-type finite elements prevent the evaluation of failure modes such as local buckling, cross-section ovalization, and detailed stress-strain distributions around the circumference of the pipe directly. They need to be established later by comparing the obtained maximum developed tensile and compressive strains to the corresponding strain limits provided by local codes.

Shell elements can be employed with nonlinear soil springs as well to overcome some of these limitations as they can evaluate the stress and strain mapping on the pipeline geometry. The internal and external pressures in this approach are uniformly distributed on the faces of the shell elements and represent the real case relatively accurate to the beam model. In the shell model, the Winkler soil springs are connected to nodes transversely, and the force-displacement relationship is evaluated based on the projected cross-sectional area in the relevant direction (Banushi, 2016).

The shell spring model was first adopted by Takada et al. (1998) to assess the continuous steel pipeline response to normal and reverse fault movement with a 45° dip angle. To overcome the limitations of computational power at their time, they delimited the length of their model to 30 times the nominal diameter of their pipe. They defined the material using elasto-plastic properties with yield stress of 366 MPa and hardening stress of 550 MPa at 5% strain. And visualized the longitudinal and circumferential strain distribution on the pipeline. As a result, they observed that fault displacement has a significant effect on the maximum value and location of axial strain on the pipeline. In addition, they proposed the term "allowable fault movement" and defined it as the fault displacement associated with the maximum allowable strain (5% for steel) for the continuous pipeline. And made a relation between allowable fault movement and various soil stiffness as well as dip angles. In conclusion, they suggested an inverse relationship such that increasing soil stiffness causes a decrease in allowable fault displacement. And allowable fault displacement varies significantly at changing dip angles. Such that for normal faulting, small angles will benefit the pipeline performance and 60° is the worst scenario. For reverse faults, however, 45° degrees is the most favorable whilst 60° is again the worst case. They also stated that continuous pipelines are more susceptible to reverse faulting than normal faulting because local buckling governs their performance. Finally, they concluded that using shell modeling for the estimation of the response of continuous steel pipelines crossing faults is advisable, specifically for reverse faults as they are more susceptible to the factor of local buckling, which beam models have limitations for its calculation.

Liu et al. (2004) are among the first to introduce the hybrid use of beam and shell elements in Winkler foundation models. The purpose of their proposed equivalent boundary method was to eliminate the requirement of excessive computational resources caused by large lengths of shell elements along the whole pipeline. To estimate the pipeline response under fault movements, their model utilizes the plastic shell elements only on pipeline segments that are significantly close to the fault. And uses elastic beam elements for parts that are not critical to the analysis. At the boundaries of the shell elements, the model utilized nonlinear Winkler soil springs as well as ap-

plied equivalent boundaries. The model was verified to be still accurate and found to be reducing the amount of computational power and time required for the analyses.

Melissianos et al. (2016) evaluated the mitigation capabilities of their proposed flexible joint design undergoing strike-slip faulting with a 3D beam on spring model. The model consists of a steel pipeline that has been enhanced with the proposed hinged bellow-type design joints to make a transition into a segmented pipeline in near-fault locations. They numerically modeled the proposed joint by a rotational spring at the center point of the joint, without considering the joint length. They point out that the reason behind the method is to concentrate strains at the joints while keeping the continuous steel parts mostly unstressed.

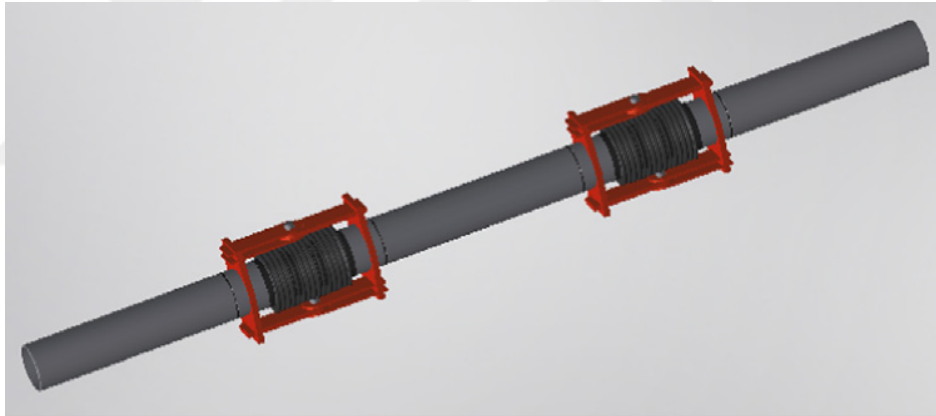


Figure 2.4. Illustration of the flexible joint design proposed by Melissianos et al. (2016).

They conducted a parametric study to observe the influences of fault crossing angle, magnitude of fault offset, joint rotation capacity, burial depth, diameter to thickness ratio, and fault trace on the seismic performance of the proposed pipeline joints. As a result, they obtained the optimum amount of these joints required as well as the most critical locations to utilize them. The findings of their study state that the proposed joint design is suitable for seismic design of buried pipelines, especially for fault crossings. The obtained strain information from the numerical model showed

that joints accommodated a significant amount of strain by rotation which would be otherwise induced on the body of the pipeline. Also, the parameters they explored showed that the influence of the joint is maximized when the fault crossing angle is fully perpendicular to the pipeline, due to the bending moment governing the response. Conversely, decreasing fault crossing angle increases the fault offset, resulting in axial tension governing the response and reducing the influence of the proposed joint-to-strain accommodation. For that reason, they suggest that hinged bellow-type joints should be used for fault crossing angles larger than 70° . As for the other parameters, they state that it does not correlate with the axial tensile and compressive response of the joint and diameter over thickness ratio as well as burial depth. Hence, they state that the efficiency of the joint has no relation with the cross-sectional geometry of the pipeline and local soil conditions. Lastly, they point out that geological conditions as well as soil properties lead to complexities in predicting the resulting fault trace. And for this reason, their suggestion is to utilize the these joints along the entire pipeline.

Wham et al. (2016) conducted two-dimensional finite element analyses to investigate responses of segmented cast iron pipelines with lead caulked joints and ductile iron (DI) with bell-spigot joints to ground deformation caused by tunneling. The tunneling in this study is represented by a 6.1 m diameter tunnel in clay and sand deposit. It is assumed that the tunnel undergoes settlement due to earthquake motion and causes local displacements in the soil underneath the pipeline. The finite element model was created on Abaqus, which utilized Winkler soil springs to simulate the surrounding soil as well as the displacements caused by tunneling. The study focused on pipelines intersecting the settlement profile perpendicular to the tunnel centerline. It also included large-scale laboratory tests to establish the joint behavior. The analysis showed that cast iron pipelines were significantly more susceptible to leakage due to joint rotation. They encountered this effect especially in pipelines crossing the tunnel centerline. Joint rotations as well as maximum tensile strains for pipelines in sand exceed those in clay by up to three times for the same geometric conditions. Their results also showed that DI pipelines have relatively more resistance against leakage in all the studied scenarios.

Banushi et al. (2020) conducted a numerical simulation of segmented pipelines using Winkler foundation model. They stated that the enlarged portion of the bell ends increases the axial friction emerging between the pipe and the soil and causes the pipe to exert less movement at that locations because of greater lateral soil pressure. They suggested that simulating this phenomenon requires complex 3D modeling approaches along with the use of contact interactions to account for nonlinearity. They also listed some of the problem-specific limitations they encountered in their study. They stated that the Winkler springs approach assumes a constant friction reaction along the body of the segments. Additionally, they suggested that the real case involves operational loads and the complexity of interface interaction at the edge of the PGD zone. The intensity of this interaction can vary in radial and axial direction due to the expansion and contraction of the hoop during the process.

Argyrou et al. (2020) created a two-dimensional (2D) FEM model in their research following the base aspects of the American Society of Civil Engineers (ASCE, 1984) guidelines). By establishing this model, they evaluated the response of segmented ductile iron (DI) pipelines to ground rupturing. The model consists of nonlinear beam elements and utilizes the Winkler soil approach on ABAQUS. Their model accounts for three cases, including the coupling of shear forces at the soil-pipe interface with forces acting normal to the longitudinal axis of the pipeline. The model also incorporates the combined effects of axial slip, pullout resistance, and joint rotation. Lastly, it includes the axial resistance of the joints to account for the friction response resulting from relative pipe-soil movement. Using the model, they simulated the response of restrained axial slip (hybrid system), push-on (unrestrained system), and fully restrained joints to strike-slip faulting. The outcome of their study compares the ability to accommodate fault offset for different fault crossing angles. They suggest that restrained axial slip joints (a hybrid system) can accommodate approximately 2–9 and 2–10 times more fault offset than push-on (unrestrained) and fully restrained joints, respectively. They also further confirmed that the seismic performance of segmented pipelines is governed by four limit states: tensile, compressive, and rotational strength of the joints as well as the buckling strength of the pipe's body. They stated that these governing limit

states are correlated with a specific range of fault crossing angles. They showed that the largest fault offsets are still accommodated by all three types of joints without failure when the angle is at 90° . Lastly, they offered that the pipeline limit state could be governed by interchanging limit states such as rotational capacity depending on the relative distance of the joint to the fault crossing.

Banushi and Wham (2020) conducted a series of nonlinear numerical analyses on hybrid-segmented pipelines under longitudinal PGD. They evaluated the seismic response as well as the deformation capacities of both typical bell-spigot jointed ductile iron (DI) pipe and axially unrestrained polyvinylchloride (PVC) pipe. In addition, they validated the proposed analytical approach in Wham and Davis (2019). The finite element model created on Abaqus is initially used to investigate pipeline performance using Winkler soil springs. Later, they determined joint displacement capacity and other parameters which influence the pipeline response. They have done this by using a semi-analytical method they proposed. A final comparison is made to evaluate the result's accuracy using the analytical approach proposed by Wham and Davis (2019). They followed the two example initial conditions of Wham and Davis (2019) approach, meaning that the geometry of the deformation zone is a block pattern, with block lengths of 91.4 and 182.9 m. They established the material properties in models within the von Mises plasticity theory with nonlinear hardening. The initial static step of their model included a pullout analysis that simulated the system's response in tension zone. As a result, they observed that the bell-spigot jointed DI pipes accommodated 2.3 times greater tensional displacement (56.2 cm) than the axially unrestrained PVC pipes (23.8 cm). In addition, they stated that the bell neck of the DI pipeline subjected to pull-out generates passive soil pressure that positively influences its deformation capacity, which is the anchoring effect that is mentioned earlier. This effect is not observed on PVC pipelines due to the symmetric design of their displacement accommodating joint restraints. The comparison between results from the proposed analytical approach and numerical models matched with little to no error. And confirmed that Wham and Davis (2019) approach is a viable method to investigate axial strains acting on the pipeline as well as its response to permanent ground deformation.

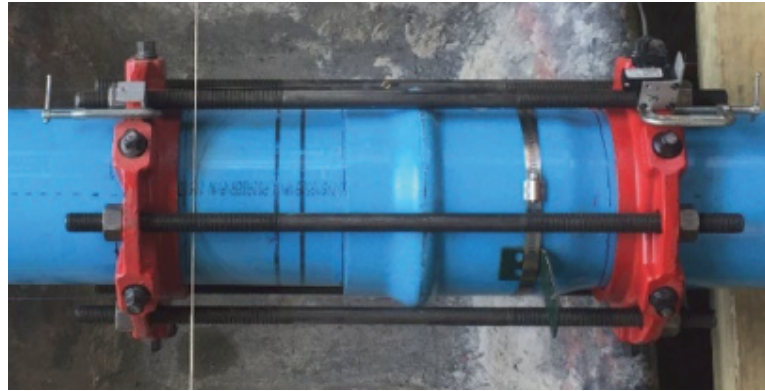


Figure 2.5. Axially unrestrained PVC joint capable of expansion (adapted from Banushi and Wham, 2020).

2.3.2.2. Solid Continuum Model. In continuum models, the most substantial difference is that the pipeline is discretized into a solid continuous elements instead of beams. The surrounding soil is modeled as 3D solid elements, and unlike the beam-type model, their contact with the pipeline is modeled as well. As the nonlinearity and complexity of other aspects of the model increase significantly, several advantages and disadvantages arise. Compared to the beam-type model, the detail in obtained results become substantially more precise, with the cost of increased need of computational power, design effort, as well as time. More specifically, the addition of more boundary conditions, resulting degrees of freedom, convergence difficulties, fault rupture modeling, and contact elements increases the required resources. Additionally, to ease the computational and modeling difficulties mentioned above, the continuum model allows the use of springs similar to the ones used in the beam-type model for modeling the soil instead of solid 3D soil blocks. However, springs in this approach are often modeled as nonlinear.

Kokavessis and Anagnostidis (2006) study was one of the first attempts to utilize the continuum soil model for pipelines. Their analysis serves as one of the foundations for research regarding continuum modeling and complex soil-pipeline interactions in pipeline systems. They created the model using ANSYS and conducted stress analysis

of buried continuous steel pipelines. The pipeline is represented using shell elements and accommodates external loads, internal pressure, load from the soft clay deposit, and traffic without internal pressure, as well as buckling cases. The mentioned traffic load is a specific case of a train crossing an overlying railroad. In addition, they simulated seismic load conditions in the case of pipeline crossing faults. They concluded the study by stating that while traditional methods can estimate pipeline stresses, incorporating contact elements enhances the ability to better represent in-situ conditions, especially under complex loading scenarios. Also, they confirmed that ground displacements such as those resulting from faulting as well as variety of boundary conditions can be simulated accurately using continuum modeling.

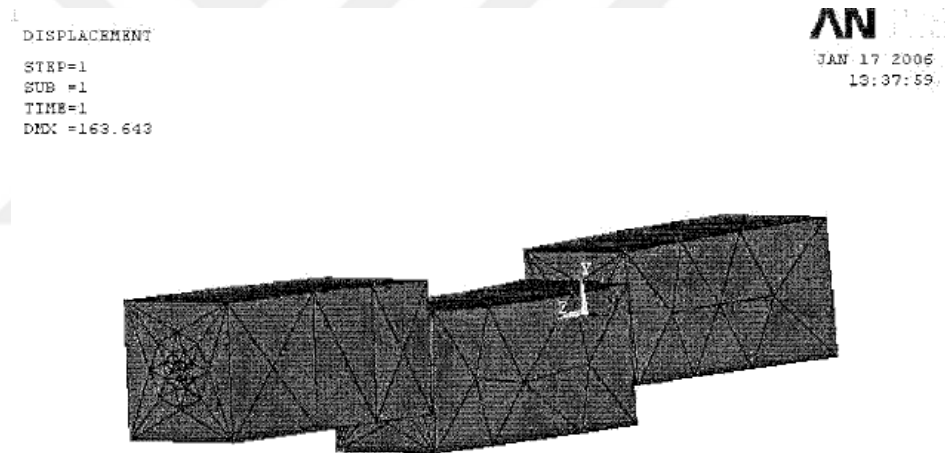


Figure 2.6. 3D continuum model of Kokavessis and Anagnostidis (2006).

Odina and Tan (2009) model can be considered among the first sophisticated continuum models. They suggested that the use of the Winkler springs approach may result in overly conservative results in certain types of environments. And established a nonlinear continuum finite element (FE) model buried in a seabed to simulate the pipeline interaction with large ground movements on Abaqus, as well as an additional beam on the Winkler foundation model to make a comparison. The focus of their study is to confirm that strains obtained from the continuum model are beneath the allowable limits with a more realistic approach. And differentiate the obtained strain

values between the two approaches. To capture the large plastic deformations on the pipeline realistically and using minimum computational power, they utilized the quadratic 20-node reduced integration brick elements (C3D20R) for the pipeline itself and the soil blocks. However, to not waste power for the segments that are further away from the fault crossing, they utilized 2-node linear pipeline elements with hybrid formulation (PIPE31H). And connected these elements by using kinematic coupling restraints. They translated one of the soil blocks by 500 mm perpendicular to the pipeline axis to simulate the strike-slip fault rupture during a seismic event. As a result, the obtained strain values showed that the continuum model calculated higher longitudinal strains compared to the Winkler foundation model. They accounted this to detailed values contained in boundary conditions introduced by the continuum model as well as the simplified assumptions of the Winkler approach. They suggest that the Winkler approach still provides a great method to preliminarily understand the potential impact of the earthquake on the pipeline during the early stages of design due to its easier implementation. However, they state that the continuum model has many more benefits, especially for the pipelines that are already built. Such that it can accurately simulate complex soil-pipeline interactions, soil movement, nonlinear visco-elastic soil properties, and heterogeneity in soil.

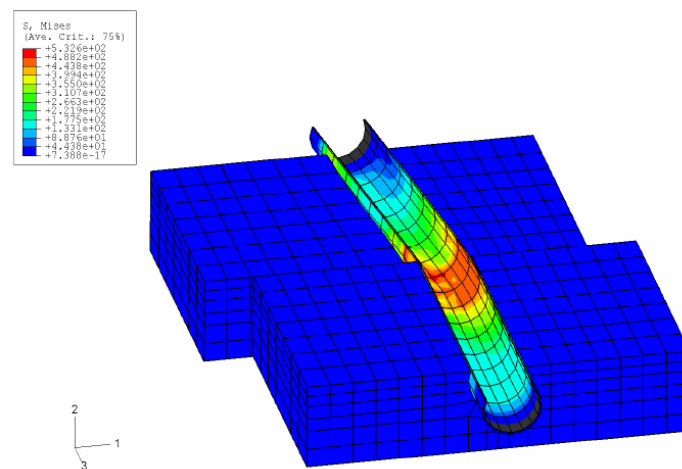


Figure 2.7. Model results showing von Mises Stress Contour plot of deformed continuous pipe (adapted from Odina and Tan, 2009).

Vazouras et al. (2010) established a finite element model for evaluating pipeline response to strike-slip fault crossing. The study investigated various parameters such as fault crossing angle, soil properties, as well as pipe deformation modes for buried continuous steel pipelines. The models were created on Abaqus and consisted of shell elements for the American Petroleum Institute grades X65 and X80 steel pipeline and plastic Mohr-Coulomb model for the soil element. The study was conducted to explore different parameters such as diameter-to-thickness ratios, materials, soil, and pipeline properties. Excluding results regarding continuous pipelines, they stated that stiff ground conditions reduced critical fault displacement and vice versa. A similar trend is observed in cohesionless soils, where pipelines buried in loose sands exhibit higher critical fault displacements compared to those in dense sands.

In their study, Wham and O'Rourke (2015) conducted a series of FE simulations on ABAQUS to characterize the behavior of the standard ductile iron push-on (TY-TON) joints with elastomeric gasket seals. By doing this, their goal was to explore how rotation influences metal binding in relation to axial pullout. The results were used to determine the magnitudes of rotation and moment required to initiate joint leakage. By performing uniaxial tension and one-dimensional compression tests on the elastomeric gasket, which they assigned with hyperelastic properties in the numerical environment, they characterized gasket behavior under extreme loading as well. Their numerical models revealed that joint leakage does not depend on the load path and a unique pressure boundary can accurately predict leakage across different deformation scenarios.

Demirci et al. (2018) developed a finite element model using ABAQUS/Explicit to simulate the behavior of buried continuous steel pipelines crossing strike-slip faults. They focused on addressing convergence difficulties often encountered in standard FEM solvers due to complex soil-pipe interactions. The study validated the explicit solver approach by simulating the lateral movement of the pipeline crossing a strike-slip fault in soft clay soil. Additionally, energy balance checks were conducted to ensure the quasi-static nature of the analyses. The explicit simulation results were compared against

standard solver outcomes and showed good agreement and confirming the reliability of explicit methods for quasi-static problems. The study highlighted that explicit solvers could effectively handle nonlinearities and complex contacts in buried pipeline systems under permanent ground deformations.

Li et al. (2024) developed a 3D continuum model of segmented cast iron pipelines with lead-caulked joints subjected to strike-slip faulting to investigate their kinematic response. The study included the post-peak softening behavior of dense sand, which was defined using modified Mohr-Coulomb plasticity model. The model represented the behavior of lead caulk typically used in between cast iron joint elements by utilizing a connector element called “busing element” that restrains the axial, shear and rotational motion of the joint elements with spring elements. The parametric analyses revealed that neglecting the strain-softening characteristics of the surrounding soil could lead to an underestimation of the peak rotational response at the joints. In addition, they stated that factors such as fault-pipe intersection angle, joint adhesion, and burial depth significantly influence joint kinematics.

Li et al. (2025) conducted a study involving 3D continuum modeling to investigate structural behavior of segmented prestressed concrete pipelines subjected to strike-slip faulting. Their study had emphasis on how fault location and crossing angle influence damage mechanisms. In results, they showed that fault planes intersecting the pipeline at angles closer to 90° led to both lateral and axial displacements. These motions resulted in material failure leading to pronounced damage on joints and circumference. They suggested that for prestressed concrete pipelines, it is not desirable for surface faulting to occur at near-perpendicular angles crossing pipeline joints and should be avoided. The study also emphasized that damage progresses through sequential material degradation, as it started with the concrete core and extended to the steel components with the increasing fault displacement.

2.3.2.3. Computational Techniques and Related Software. The numerical models can be based on three computation techniques: The finite element method, the finite dif-

ference method, and the discrete element method.

Finite Element Method (FEM) is a numerical computation method that is used to solve engineering problems in various fields. Particularly for civil engineering, the method is utilized for structural analysis and soil-structure interaction problems, etc. However, they are capable of running thermal, fluid dynamics, electromagnetics, acoustics, etc. analyses as well. The principle of FEM is that it involves breaking down a complex problem (in this case structure or system) into smaller, simpler parts called finite elements. These elements are interconnected at points called nodes and assigned with material and geometrical properties. Then, for each element, the governing equations are derived by formulating the stiffness matrix, mass matrix, or other relevant matrices for each element. These relatively simpler equations are later assembled to form the global system of equations that represents the ultimate problem. Lastly, boundary conditions are applied, and the solution is then approximated over the elements utilizing simpler mathematical equations. Commonly adapted FEM-based software that is used in this regard are SAP2000, ABAQUS, ETABS, ANSYS, PLAXIS, etc.

Finite Difference Method (FDM) is a computational method that relies on discretization. The continuous domain (e.g., time or space) is divided into a finite number of discrete points called grid points. Then, the unknown variable in the differential equation is determined at each grid point by approximating the derivatives between adjacent points. FDM has two disadvantages compared to other available computation techniques. One is the limited information on non-discretization points. The method solves for the function f , at the grid points. As a result, the values of f at points in between the grid points are not explicitly known and are approximated based on the values at neighboring points. Meaning that any information on the solution between these discrete points is not directly available. Which could lead to inaccuracy in results if the solution varies significantly between grid points. The second major deficiency of FDM is its restriction to grid geometry. It inherently relies on a structured grid, which is typically rectangular or cubic. This restriction limits its ability to handle complex geometries such as curved boundaries or abstract shapes. As these shapes are

estimated with a large number of very fine rectangles or cubes, the required computational power increases. Often utilized in advanced geotechnical analyses, common FDM software are DEEPSOIL, FLAC, etc.

Distinct Element Method (DEM) is a computation technique that is based on simulating the contacts and interactions between particles. As opposed to other methods, elements are discontinuous and are composed of each distinct particle with a unique quantity instead of a single solid. It is used to analyze engineering problems regarding granular materials such as those in geotechnical engineering. The common software that utilizes this method are LS-DYNA and PFC.

2.3.3. Laboratory Experiments

Laboratory tests are reliable ways to evaluate the seismic performance or response of engineering structures. They account for in-situ conditions that are usually not simulated in numerical modeling or analytical approaches such as heterogeneity or deformations in ground. For pipelines specifically, many types of experiment methods can be used to establish a variety of properties. However, as this assertion concerns numerical computation, only the basic working principles behind them are discussed briefly along with previously done tests.

The commonly carried out experiments for pipelines are hydrostatic tests, four-point bending tests, pullout tests, compression tests, external loading tests, torsion tests, and large-scale split box tests. Hydrostatic tests are done to determine the strength and leak prevention of the pipeline by filling it with water and pressurizing it beyond its operational capacity. They are done to ensure the pipeline can handle maximum pressure without bursting. The purpose of four-point bending tests is to evaluate the stiffness, flexibility, and bending strength of the pipeline. They are executed with a similar setup that is used to test concrete beams, however with four supports instead of three to avoid directly applying stress on pipe joints. Pullout tests are one of the critical tests done on segmented pipes as they are highly prone to pulling

apart from each other. The pipe is fixed from opposite ends before introducing the tension to determine the amount of leakage. The compression test setup is similar to pullout tests, only with compression instead of tension. Using these tests, telescoping or crushing failure near the joint is evaluated. External loading tests are done to test the pipeline's resistance to external factors, such as those exerted by soil or water. The pipe is subjected to various distributed loads from its exterior to determine the hoop strength and establish the failure criteria. Torsion tests measure the pipeline's ability to withstand rotational forces that might occur along its longitudinal axis due to complex ground movements. They are often done on continuous pipelines as they lack the ability to rotate freely using joints. Large-scale split box tests are designed to simulate fault rupture across the buried pipeline. The pipes are buried in a box divided into two halves where one side can be moved horizontally or vertically depending on the characteristics of the event while the other side is kept stable. This allows various parameters to be studied such as local site conditions, faulting, etc.

In addition to their FE analyses, Wham and O'Rourke (2015) conducted laboratory four-point bending experiments of the same TYTON joint used in their numerical models. The experiments focused on obtaining the magnitudes of rotation and the moment at which a joint leakage would initiate. Results from their experiments showed that the said relationship between joint load and leakage establishes a pressure boundary for the leakage in DI joints. Which establishes a limit on the mechanical behavior of joints responding to ground deformation. Additionally, they confirmed that to analyze the mechanical behavior of push-on joints under extreme deformation, leakage needs to be considered as a function of axial pullout and rotation together. Which was previously done using only the rotation up to a point where metals bonded together. The experiment also showed that when the spigot was fully seated into position, the joint gained its maximum capacity to allow rotation without leaking until 16° . To prevent the irrecoverable and high levels of local damage applied to the spigot, they suggested that this limit for joint pullouts up to 20 mm joint depth.

Melissianos et al. (2017) investigated and compared continuous pipes with bellow jointed segmented pipes response to fault rupture by conducting three-point bending experiments. They studied the induced strains due to faulting as well as explored the deformation behavior of the said pipes. The experimental setup involved inducing transverse displacement at the midpoint of the pipes while maintaining the ends fixed. The setup is aimed to replicate the S-shaped deformation pattern typically observed in buried pipelines subjected to strike-slip fault rupture. They expected critical bending moments to be located at the fixed ends and the midspan and placed the flexible joints strategically near these regions to maximize their effectiveness in reducing the strains. As a result, they pointed out that the inclusion of bellow joints resulted in a significant reduction in both tensile and compressive strains compared to continuous pipelines. The method effectively minimizes the chances of local buckling and tensile fracture. In addition, they observed that the flexible joints exhibited non-linear behavior during the bending tests and could accommodate significant rotation and axial movement before failure.

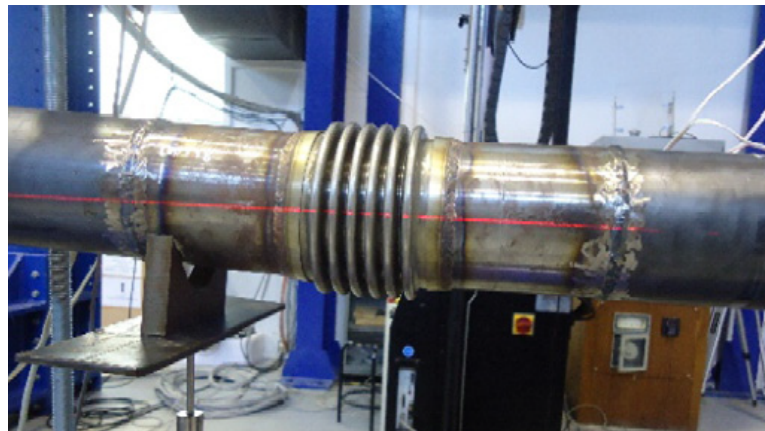


Figure 2.8. Bellow jointed segmented pipe (adapted from Melissianos et al., 2017).

Oda et al. (2017) conducted a series of both four-point bending tests and large-scale split box tests to establish the performance criteria of buried segmented ductile iron pipes. The nominal diameter of the pipeline is 150 mm and consists of a typical

bell-spigot design with a locking mechanism. Both experiments were done with internal pressure of 0.55 MPa. They performed four-point bending tests for a maximum deflection angle of 32° . The bending moment is gradually increased under pressurized conditions up to 550 kPa while keeping the internal pressure at 0.55 MPa throughout the experiment. The results of four-point bending tests showed no leakage occurred at rotation angles of up to 12° , which is 1.5 times the performance limit of their local code. The first leakage occurred at 12.2° with roughly 0.41 liters per minute. They stated that leakage became significant at angles of 14.3° with roughly 26.5 liters per minute and stopped at 16.6° , later followed by a stabilized pressure. By conducting a series of large-scale split box tests, they also simulated the effects of fault rupture on DI pipelines. They buried the pipe 76 cm down in a compacted sand environment with a fault crossing angle of 50° , and fixed both ends to simulate an in-situ pipeline. According to their results, the pipeline absorbed fault displacements as much as 1 m, and pullout followed by leakage was observed at 1.13 m. And successfully fulfilled the safety margins of DI pipelines against joint pullout.

In their large-scale laboratory tests, Wham et al. (2017) investigated and characterized the soil pipeline interaction as well as the response of pipelines to varying cases of PGD. They obtained the axial force and displacement performance of earthquake-resistant ductile iron pipelines buried in sand. Moreover, they explored the pipeline diameter's influence on the pullout resistance of the joint. The DI pipe they tested had a 150 mm nominal diameter and the cross-sectional area of the segment was 241 cm^2 . The bell part had a 13% enlarged cross-sectional area (435 cm^2) compared to conventional DI pipes due to the specific commercial pipe they chose. They joined the two pipe segments together and placed them in partially saturated sand with a burial depth of 760 mm. And compacted the surrounding sand to simulate a medium-dense to dense granular backfill. The results of laboratory tests confirmed that axial slip occurs when soil resistance on both parts of the joint exceeds the force required to pull the joint apart. They further proposed that the anchoring effect caused by enlarged bells in segmented pipelines contributes significantly to pullout resistance. Specifically, they stated that the flat face of the bell generates higher pullout resistance and reacts

more stiffly compared to the curved face. Another finding of their study is that higher forces are needed to initiate pullout in denser soils, as it increases the joint resistance. They observed an increase in this effect particularly at small displacements. Lastly, they stated that the most significant finding is the load drop across the enlarged bell joints reaching 20-24 kN, as well as the similarity of maximum axial resistance between the bell and the segment. This means that axial forces at the bell can match those along the pipe's straight sections, but full mobilization of shear along straight sections requires less movement.

Wham et al. (2018) further investigated the same concept in Wham et al. (2017) for axially unrestrained PVC pipelines. And obtained force-displacement relationships to establish the strength as well as pullout capacity of restrained pipe joints. Moreover, the type of joints used in this setup are restrained in every direction except the longitudinal axis. In their results, they observed that PVC pipelines have great tensional strength in the axial direction as no failure was observed. They also observed the significance of burial depth on the pullout resistance of restrained joints. Specifically, for displacements above 610 mm, they increased the burial depth by two times and observed up to 115% increase in axial resistance. They stated that the force-displacement relationship is nonlinearly increasing at relatively large longitudinal displacements.

Demirci et al. (2021) investigated the behavior of buried continuous pipelines subjected to strike-slip faulting through a combination of experimental testing and 3D finite element (FE) modeling. They derived relevant scaling laws and non-dimensional parameters to better understand soil-pipe interaction during fault movements. The study included four physical model tests and validated the numerical model against experimental results and a real-world pipeline failure case from the 1999 Kocaeli Earthquake. A detailed parametric study was conducted considering various factors such as fault crossing angle, relative soil-pipe stiffness, pipe slenderness (D/t), burial depth, and pipe end conditions. Their findings emphasized that the longitudinal strain in pipelines under strike-slip faulting is highly dependent on these parameters, and that both experimental and numerical approaches are crucial for understanding and im-

proving pipeline resilience against fault-induced deformations.

Liu et al. (2023) conducted laboratory centrifuge tests on segmented DI pipes in order to simulate the response to strike-slip faulting. Their test involved burying a 2.6 m diameter DI pipeline consisting of five segments in a soil box and applying ground displacements at a centrifugal gravity of 30.7 g. The corresponding ground displacement was pointed out to be 60 mm to simulate a 1.84 m long strike-slip fault. As a result, they obtained the induced pipe strain as well as the response of the joints. Their findings revealed that the main response of buried segmented pipelines undergoing strike-slip fault displacement is governed by the joint deformation, with joints that are closer to the fault being most prone to damage. Also, the applied strain and resulting joint deformation on the pipeline are both nonlinearly affected by the fault displacement. And the distribution of joint deformation is not symmetrical. The maximum and minimum strains were located at a point with some distance to the fault trace, where the bending moment was found to be the highest along the pipeline.

2.4. Summary of the Literature

Buried segmented pipelines are prone to failure due to PGD or TGD-induced excessive loads or ground displacements caused by earthquakes. A review of studies analyzing pipeline failure data from the aftermaths of past earthquakes suggest that PGD poses a greater hazard to pipeline performance. Additionally, it is possible to interpret the lack of studies considering the TGD induced effects on pipelines to the generally accepted idea that passing waves pose a lower hazard on pipeline performance. The body of soil surrounding the pipeline (or even in some cases the pipeline itself) goes beyond its elastic limit and undergoes plastic deformations. On the other hand, a pipeline may also fail due to other reasons caused by excessive ground deformation. Three failure modes can be associated with segmented pipes as they are: axial pullout, crushing of bell or spigot, and circumferential failure.

In-depth analysis of the response of buried segmented pipelines is now possible throughout the various state-of-the-art computing software. To be able to simulate and analyze these failure modes numerically, the Finite Element method can be utilized as well as Finite Difference or Discrete Element methods. Although each of these methods can simulate nonlinear behavior in materials, geometry, and boundary conditions, the Finite Element method is generally considered more robust and versatile compared to other more “brute” methods. Abaqus/CAE 2024, being able to handle nonlinearities in both the pipeline or soil material and the geometry, is found to be a versatile and reliable finite element software to establish and analyze complex nonlinear continuum models.

Determination of the ground strain leading to axial pullout or joint crushing is vital and requires realistic nonlinear numerical modeling, which is done either using Winkler soil springs or 3D continuum modeling. Solid 3D continuous models include much more detail compared to the soil spring method. However, the process is complex and requires a skilled analyst as well as time.

The segment and joint properties such as strength, stiffness, geometry, and material properties, as well as the properties of the soil and its interaction with the pipeline must be modeled with utmost detail. However, as mentioned previously, conducting a detailed analysis where soil movements, nonlinear visco-elastic properties of soil, heterogeneity in soil, and soil pipeline interactions are captured is complex. High-precision numerical models are overly sensitive to inaccurate data, which may lead to significant differences between results.

2.4.1. Identified Research Gaps and Limitations

In literature, there are many studies that utilize 3D continuum modeling in FEM simulations of buried pipelines. It is observed that the PGD cases (e.g., faulting, liquefaction) are more commonly studied for buried pipeline systems, as these deformations can result in more localized and higher stress and strain accumulations compared to

the effects of TGD. However, the majority of these studies are focused on continuous pipelines, which don't require 3D continuum modeling of the pipeline itself due to availability and simplicity of 2D shell elements. Instead, these studies often model the soil domain exclusively as a 3D solid body only.

On the other hand, studies that focus on FEM analysis of PGD or TGD induced buried segmented pipeline performance are scarce. These types of simulations (in addition to those of continuous pipelines) require careful definition of the joint geometry and its kinematics using various methods. The geometry of a joint mechanism cannot be truly modeled using shell elements and can only be introduced to the FEM environment through detailed solid 3D models. Additionally, the finite elements need to be specified with smaller spaces between nodes to represent the true geometry of the joint mechanism, which causes a significant increase in required computational resources relative to continuous pipeline studies.

It is revealed that the introduction of pipe segmentation (therefore complex joint geometries) to FEM analysis of buried pipelines brings many considerations to the modeling process. And the true geometry of the pipeline is expected to significantly influence the FEM results. However, since the widely accessible computer power and FEM tools have only developed rapidly in the last two decades, the capability to solve highly specific problems (such as the 3D continuum model for pipelines) have only recently started to be incorporated into buried pipeline studies in literature.

In summary, creating realistic models for buried segmented pipelines that are thoroughly considered from all aspects requires computational power, time, and an experienced analyst. It is observed that such studies are not commonly available, and there are very few in literature.

3. PRELIMINARY VALIDATION STUDY

3.1. Validation Analysis

It is essential to confirm the credibility of numerical studies by comparing their methods with other models, laboratory experiments, or analytical approaches done by trustworthy sources. For this reason, preliminary validation models can be developed before starting a study. In the case of numerical modeling of buried segmented pipelines, the aim is to verify that the pipeline, its connection, as well as other components of the model are modeled geometrically correct, the material properties are appropriately defined, boundary conditions are represented accurately, and the interactions of the pipeline system with itself as well as with its environment are implemented. This process is critical in terms of ensuring that future simulations in the study are realistic and closely approximate the real-life scenario. In that regard, this assertion takes the laboratory experiment as well as the FEM simulations done by Wham and O'Rourke (2015) as basis for validating the upcoming numerical analyses against their valid data. The following part of the subchapter reviews the established preliminary model as well as its comparison with results of the study of Wham and O'Rourke (2015) for validation.

3.2. Laboratory Experiment and FEM model by Wham and O'Rourke

In their 2015 journal article, Wham and O'Rourke did a series of numerical and experimental research on segmented ductile iron pipelines. Their pressurized four point bending experiment consisted of several parts. The setup included two typical bell-spigot jointed DI pipes which were cut down from their original length of 5.5 m into 1.83 m and joined together to create a 3.7 m long specimen. The pipeline featured an inner diameter of 160.6 mm, outer diameter of 175 mm, and wall thickness of 7.2 mm at the main body of the segments. The DI material in the study refers to the tensile tests on coupons Wham and O'Rourke cut from the unused parts of the pipe specimens, of

which the resulting stress-strain curves are presented in Figure 3.1 below.

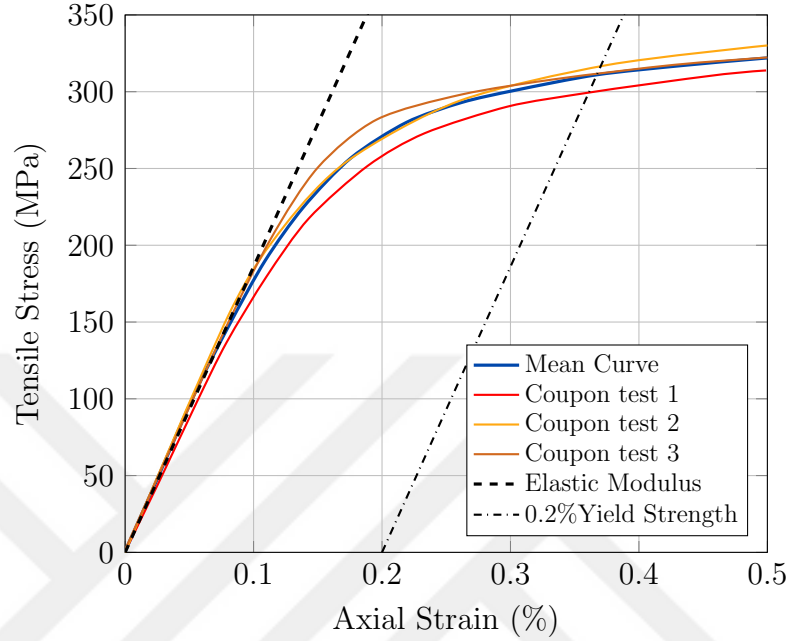


Figure 3.1. DI pipe stress-strain curves from tensile coupon tests and mean curve used in the numerical models (based on Wham and O'Rourke, 2015).

The gasket they used consists of two parts (heel and bulb) made of styrene butadiene rubber with different hyperelastic properties and joined together during the manufacturing phase. They characterized these two rubber gasket materials on Abaqus by using the data from the previously done laboratory uniaxial tensile as well as one-dimensional compression tests provided in Figure 3.2 below.

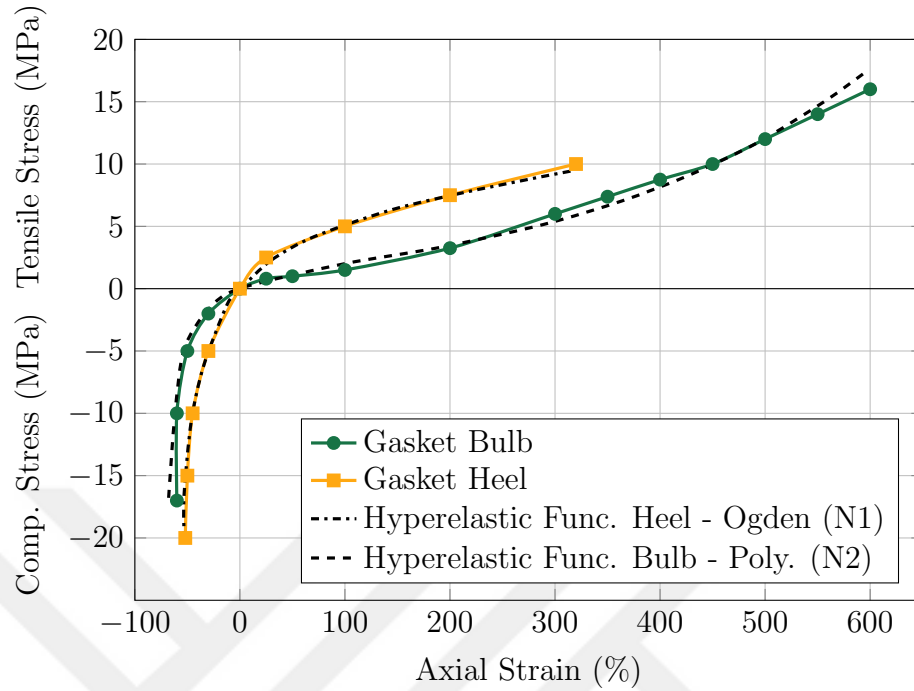


Figure 3.2. Stress-strain relationships of the gasket bulb and heel generated from material testing, hyperelastic curve fitting, and ABAQUS material evaluation (based on Wham and O'Rourke, 2015).

The geometrical and material properties of both the gasket and the joint used for validation of this study followed U.S. PIPE's regulations for TYTON joints and gaskets, same as the Wham and O'Rourke (2015) study (originally cited as U.S. Pipe, 2013a, 2013b). Their setup is rigged to induce 1800 kN of load to the pre-pressurized system through two spreader beams located above loading saddles for both specimens at a constant load rate of 0.44 kN/s. The internal pressure of the system is adjusted to 380 ± 35 kPa and maintained in that range throughout the analyses. Using various measuring devices, they obtained the vertical as well as axial joint displacements and calculated the relative joint rotations. Later, they verified the deflection of the specimen and their associated curvature. In addition, they measured the axial and circumferential strain values using strain gauges placed at the body and the crown of both specimens. Using the obtained values, they associated joint rotations with corresponding axial strains at the crown and invert, as well as the joint leakage.

3.3. Setup of Equivalent Numerical Model

Among other things, realistic numerical modeling requires establishing the geometric properties of the segment, bell, spigot, and elastomeric rubber gasket. To represent the solid 3D pipeline geometry most realistically, the parts throughout the analyses, including those for the validation, are modeled using computer aided drawing software SolidWorks with detail before imported to Abaqus. The reason behind this is that, although Abaqus is also capable of modeling solid 3D elements within itself, it is not a suitable choice for modeling in this study. Moreover, its own tools are insufficient to draw complex geometries that require high smoothness and precision.

The geometry of the DI pipe elements included in the setup were modeled after the detailed cross-sectional schemes provided by Wham and O'Rourke (2015). The gasket geometry is based on the general specifications provided by U.S PIPE and adapted to the dimensions of the 6-inch TYTON pipe. After modeling the DI iron pipes, rubber gasket, and loading saddle parts on SolidWorks in detail, they are imported to Abaqus, and their sections are defined.

Next, their material properties are defined. The hyperelasticity of gasket parts is set to be calculated in the analysis using a specialized meshing element "C3D8RH" selected from the three-dimensional solid element library of Abaqus. The C3D8RH, which is also utilized by Wham and O'Rourke (2015), is an 8-node, hybrid with constant pressure, linear brick element with reduced integration and hourglass control. The difference between hybrid with constant pressure and regular elements such as "C3D8R" is that hybrid elements interpolate the displacement and pressure stress solutions independently (Wham and O'Rourke, 2015). This allows the element to accurately calculate the strain values of materials that are nearly incompressible. Following the approach explained in the study, the results from uniaxial tensile and one dimensional compression tests were incorporated into Abaqus as user-defined test data, and the material is assigned with Poisson's ratios of 0.495 to capture its nearly incompressible behavior. To model the hyperelastic behavior of the materials, the 1st order

Ogden strain energy function (Ogden, 1972; Ogden et al., 2004) was employed for the stiffer heel component. According to Wham and O'Rourke, this function offers a good balance between accuracy and computational efficiency. For the gasket bulb, a second order reduced polynomial function (Rivlin and Saunders, 1951) was used to provide a fit to the experimental data while effectively capturing large deformations at smaller strain levels. The two styrene butadiene rubber material models were evaluated to ensure they fit with the chosen functions and are stable in the expected strain range. Their fit with the two functions are provided in Figure 3.2 above.

DI used in the setup is modeled with regular C3D8R element and defined using the stress-previously mentioned strain curve data provided by Wham and O'Rourke in Figure 3.1 above. Poisson's ratio (ν) and modulus of elasticity (E) for DI iron is selected as 0.29 and 186000 MPa respectively.

Next, the parts are assembled according to the setup provided in the study to represent the initial state. On this state, contact interactions such as those of DI to DI and DI to rubber gasket are defined. The friction coefficient between ductile irons is opted for 0.65 whilst for DI – rubber gasket, it is neglected. This is because of the excessive nonlinearities introduced to the problem by the combination of the already highly nonlinear hyperelastic behavior and the contact algorithm of the solver, which multiplies the required time greatly.

Meshing is done using hex elements and 3 mm spacings for the critical sections of both pipe specimens. For the gasket however, this value is selected as 1 mm to prevent negative eigenvalues arising from excessive distortions of the rubber during its peak deformation.

Final adjustments were applied to optimize the model, which allowed the saved computational power to be spent on finer meshing in critical regions and obtain more accurate results. In this regard, the desired outputs that are going to be calculated during the analysis procedure are carefully selected to avoid calculating for the unneeded

components (e.g., accelerations) as well as the regions. Also, the axially symmetrical geometry of the system allowed simulating only one half of the model. For this reason, the model was cut in half and symmetric boundary conditions were applied to longitudinal cross section of the system, which theoretically reduced the required computational power down to half.

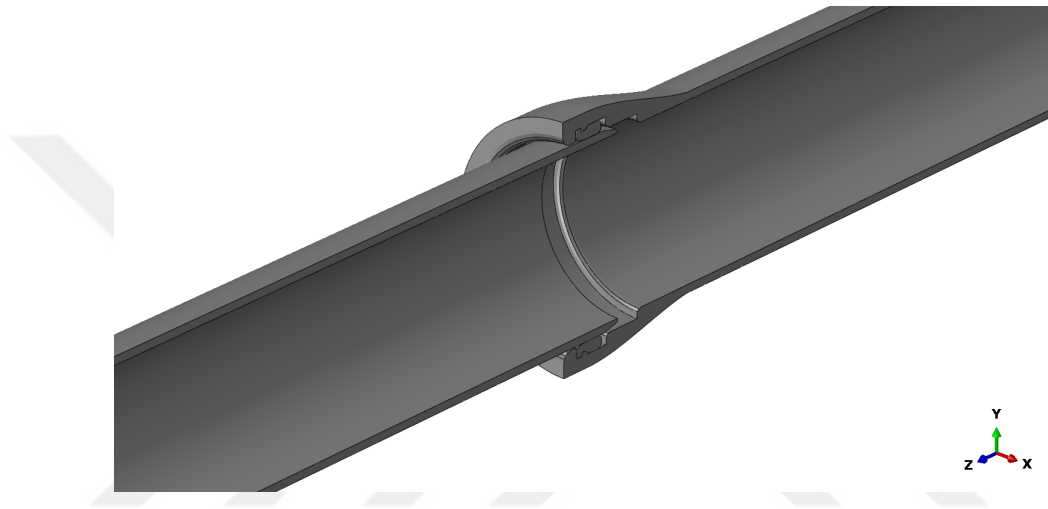


Figure 3.3. Rendering of the half-symmetry pipe joint model composed of over 460000 3D solid elements and 1.6 million degrees of freedom.

Unlike the studied model, the simulation procedure is configured in three steps instead of four because it neglects the initial axial displacements other than the case of half-insertion that is accepted as the neutral laying condition. The first step is a dynamic implicit step that includes numerical insertion of the spigot into the bell by 23 mm and the simulation of resulting deformations on the gasket elements. This step, along with the final step, is modified to work as quasi-static, meaning that the problem is solved as a slow sequence of static conditions over time. This solution method was adapted because the time periods of the predicted deformations are gradual enough that dynamic effects like inertia or damping can be assumed as irrelevant.

In the second step, the internal surfaces of the pipe along with some parts of the gasket bulb that are exposed to water pressure are defined, and the system is set to gradually introduce the evenly distributed internal water pressure until it reaches 380 kPa. Similar to the Wham and O'Rourke (2015) model, the pressure introduced negligible amounts of elastic strain on the DI parts whilst straining the rubber bulb by nearly 10

The final step includes the main four-point bending action. Since the surface dimensions of the applied load in the laboratory environment are unknown, the load is numerically simulated by applying vertical displacements in the form of a half-circle at the support locations until the desired relative rotational angle of 16° is reached. The flexible joint element that is used in the reference study is also introduced in this stage. This element, shown in Figure 3.4, prevents the axial separation of two specimens during four-point bending motion by a hinge mechanism that is clamped around both specimens, which only allows rotation on the horizontal axis that is normal to the pipeline axis.

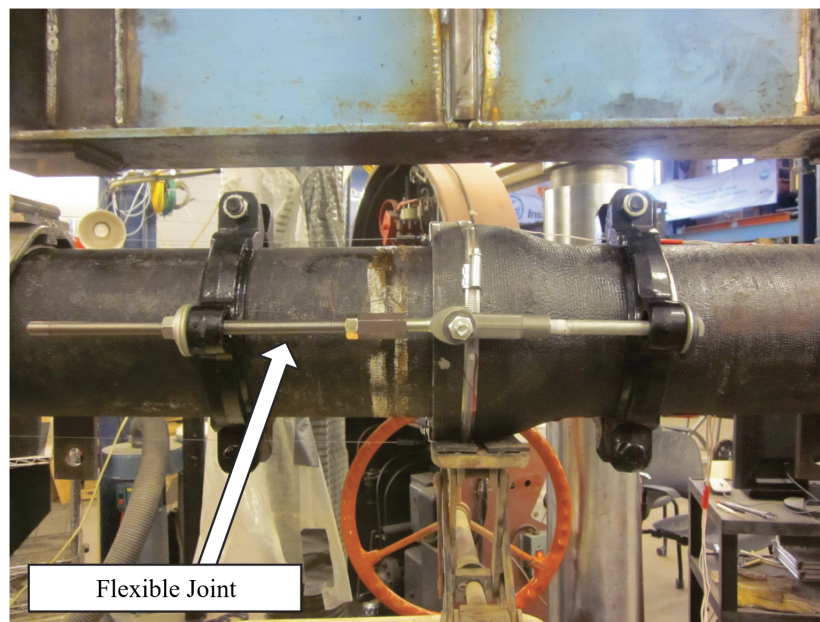


Figure 3.4. Adapted picture of the flexible joint element used by laboratory experiments of Wham and O'Rourke (2015).

Implementing this element in Abaqus through defining the same hinge mechanism with simplified elements was deemed challenging because they could not be identified in the Abaqus interface, or because of their potential absence in the software. For this reason, another method that could simulate the same kinematics was improvised and adapted. Instead of a hinge element, the mechanism is implemented to FEM environment by kinematically coupling the two nodes located on the pipeline centerline to the corresponding circumferential nodes first. Which simulated the clamps around the pipe shown in Figure 3.4 and allowed the circumferential nodes at the two cross-sections of the pipeline to yield equivalent translation during pullout. By doing this, the pullout motion is essentially governed by the two nodes in centerline of both pipes. Then, these two controlling nodes are tied together using a wire element that only works translationally in the longitudinal axis, which fundamentally simulated a “rope-like” behavior. The wire element is defined with an upper bound length restriction in longitudinal axis using “stop” condition that initiated after certain length due to pullout is reached. This improvisation prevented the two nodes, and ultimately the joint, from separating during bending on the pipeline axis.

3.4. Comparison of the results

The conducted four-point bending experiment by Wham and O’Rourke (2015) was mainly concerned with establishing a relationship for axial displacement and joint rotation to corresponding joint leakage. However, due to limitations of computer environment, evaluating leakage in this study is considered infeasible. Because of this reason, the objective of the preliminary validation is defined as matching the resulting axial and rotational values with corresponding strain data at critical areas of the system, as well as the maximum principal strain propagation on the joint geometry. The strain values on the critical regions of the pipe crown as well as the invert are compared for rotations of up to 16° are presented in Figures 3.5 and 3.5 respectively.

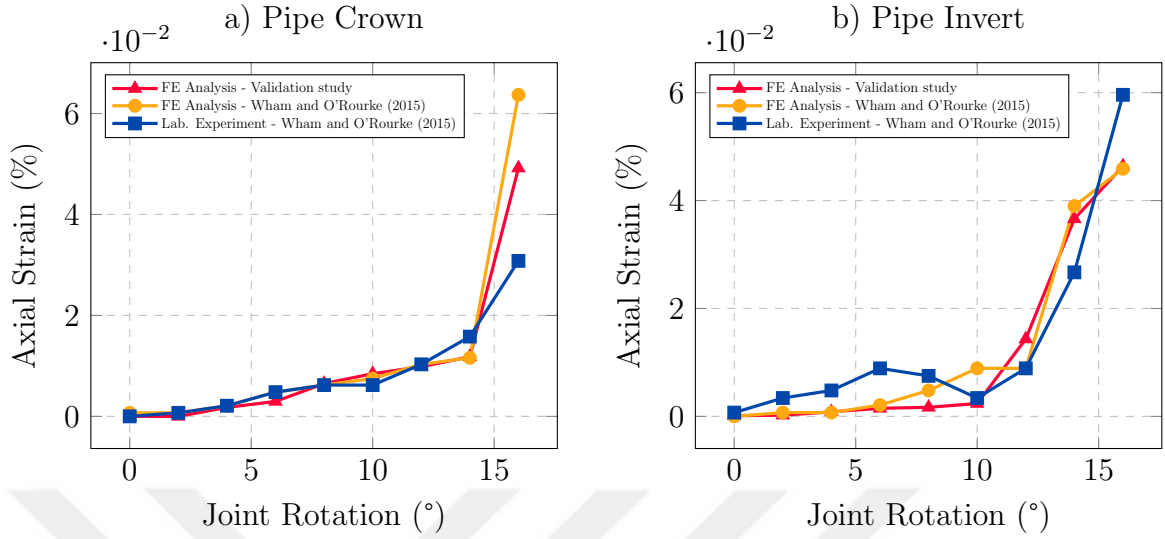


Figure 3.5. Axial strain comparison between the validation model, Wham and O'Rourke (2015) FE model and laboratory experiment measurements at a) pipe crown and b) invert.

By looking at the results in Figure 3.5a, it can be observed that the axial strains forming at the crown for the validation model follows both trends of Wham and O'Rourke study until around 14° of joint rotation. Although FE and laboratory experiment results of the selected study differ from each other after this point, the values of established FE model for validation remained between them until the final rotation of 16° is reached and significant plastic strains on ductile iron occurred. As for the pipe invert, both FE models gave similar results as they followed the same trend apart from the negligible difference (approximately 0.01%) at 10° of rotation. Whereas the axial strains from the laboratory experiment deviate from the FE models before that point. At 16° of joint rotation, the maximum axial strains observed on the FE model reached 0.0492% and 0.0464% at pipe crown and invert respectively.

For DI, it is observed that the material shows linear stress accumulations until around the point of 200 MPa as per the tensile coupon test data from Wham and O'Rourke (2015) suggests in Figure 3.1. And the relatively high nonlinear behavior of DI beyond the 200 MPa region continues until the point of around 300 MPa , beyond

which the nonlinearity decreases and the material has a broad curve until it reaches 460 MPa. These effects of DI is observable in the simulation, meaning that the material is defined correctly to the FE environment.

In addition, a visual comparison on Figure 3.6 is made between the maximum principle logarithmic strain distributions obtained at the end of the two studies. By looking at the patterns available at longitudinal cross sections, it can be observed that a high degree of similarity is present. The regions of peak strain concentration align in both models at the joint interfaces, where most significant deformations occurred. This similarity of both strain localization and intensity further reinforces the validity of the FE model in replicating the mechanical behavior and deformation characteristics of the joints as seen in the reference study by Wham and O'Rourke (2015).

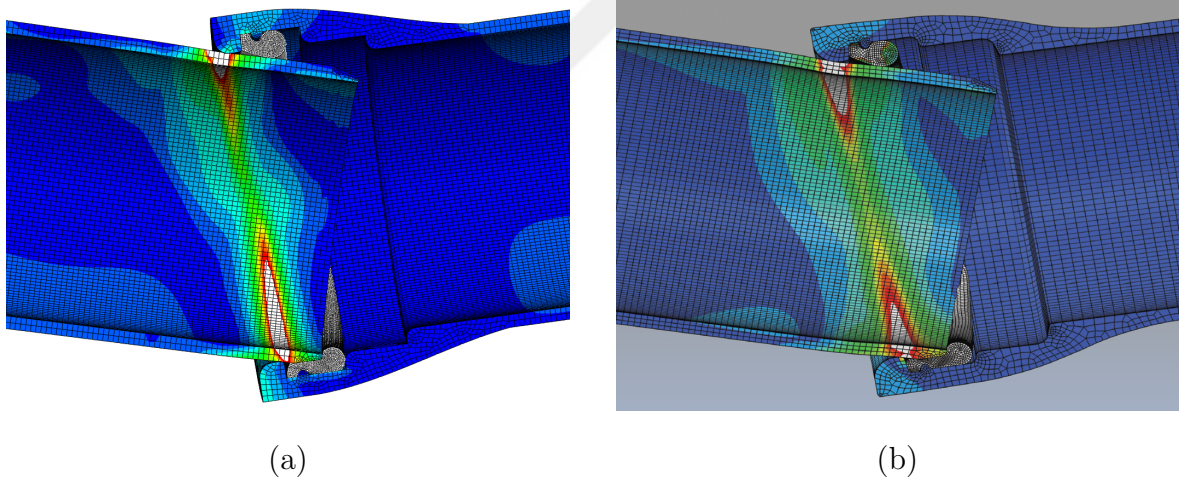


Figure 3.6. Cross-sectional view of the final stage of the bending process and the resulting maximum principle logarithmic strain distribution for (a) Validation Model
(b) Wham and O'Rourke (2015) model.

A conclusion can be made from these comparisons, as the axial strains of the established FE model strongly match those of the Wham and O'Rourke (2015) study both numerically and in terms of propagation. And the FE element analyses that are going to be conducted in the main part of the assertion represent the real nature of segmented pipelines well.

4. PARAMETRIC NUMERICAL SIMULATIONS

4.1. Studied Parameters, Cases, and Selected Tools

As discussed previously, there are many parameters which can significantly influence the response of buried segmented pipelines to earthquake ground motion. Among these, PGD induced deformations on pipelines are generally considered a greater hazard to their infrastructure. Moreover, strike-slip faulting phenomena is among the possibilities that could result in PGD on a site at which a pipeline system passes through.

In this study, three strike-slip faulting induced PGD parameters are selected as the Fault crossing angle (β), Fault Crossing Position (γ), and Fault Displacement Vector (δ), along with two soil models that represent soft clay (SC) and medium dense sand (MS), which are defined using Mohr-Coulomb plasticity model. Using the combination of these parameters, strike-slip faulting phenomena is simulated on 20 FEM models to incrementally apply a total of 1 meter of fault displacement at the rate of 5 cm/s in each case. Table 4.1 shows all the case combinations studied throughout the thesis.

Table 4.1. Different combinations of the studied parameters and resulting twenty cases.

Case No.	Fault Crossing Angle (β°)	Fault Crossing Position (γ)	Fault Crossing Vector ($\pm\delta$)	Soil Deposit
1	90	0.5	n/a	Soft Clay
2	90	1	n/a	Soft Clay
3	90	0.5	n/a	M. Dense Sand
4	90	1	n/a	M. Dense Sand
5	67.5	0.5	+	Soft Clay
6	67.5	1	+	Soft Clay
7	67.5	0.5	+	M. Dense Sand
8	67.5	1	+	M. Dense Sand
9	67.5	0.5	-	Soft Clay
10	67.5	1	-	Soft Clay
11	67.5	0.5	-	M. Dense Sand
12	67.5	1	-	M. Dense Sand
13	45	0.5	+	Soft Clay
14	45	1	+	Soft Clay
15	45	0.5	+	M. Dense Sand
16	45	1	+	M. Dense Sand
17	45	0.5	-	Soft Clay
18	45	1	-	Soft Clay
19	45	0.5	-	M. Dense Sand
20	45	1	-	M. Dense Sand

Fault crossing angle (β) defines the inclination of the fault line relative to the longitudinal axis of the pipeline orientation. It is a critical parameter in strike-slip induced PGD for pipelines as it is often regarded as among the main factors of their deformation patterns and stress distribution. Theoretically, a small crossing angle can result in predominantly axial forces that could result in pullout, while a larger crossing angle may induce bending and shear stresses on the segment or the joint, leading to

complex deformation modes. Another parameter studied is the fault crossing position (γ), which indicates whether the fault plane intersects the middle of the pipeline segment or the joint. The value is obtained by dividing the length of the pipe in the fixed side (L_f) to the total length (L) of the pipe specimen (i.e., $\gamma = L_f / L$). This positional relationship has an effect on the localized bending and axial forces, especially given that pipeline joints are the structural weak points. If the $\gamma=1$ (or 0), the stresses are expected to concentrate on the joint, whereas if $\gamma=0.5$, meaning that fault plane is crossing the middle of the segment, the resulting forces are expected to be distributed along the segment and eventually the joints. Fault displacement vector ($\pm\delta$) defines the direction of the slipping soil block. Its effects are also evaluated since δ determines whether the pipeline will experience axial extension or compression, which is expected to be critical in terms of deformation modes and overall pipeline performance similar to β and γ . In this study, the positive direction of the fault displacement vector ($+\delta$) denotes the orientation that induces tension to the pipeline in its longitudinal axis, while its negative direction ($-\delta$) denotes the fault movement compressing the pipeline. The visual demonstration of β , γ and δ is shown in Figure 4.1 below for a single example case.

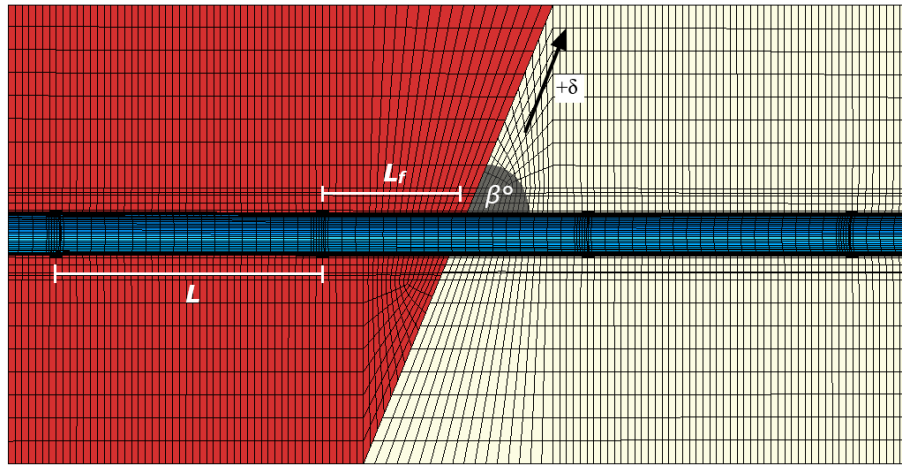


Figure 4.1. Illustration of the studied geometrical parameters, including the fault crossing angle (β), the fault crossing position ($\gamma = L_f / L$), and the direction of the fault displacement vector δ .

Abaqus/CAE 2024 software was chosen for this study due to its capability in handling complex nonlinear problems with variety of solver options. Specifically, its dynamic explicit solver is well-suited for simulating effects of earthquake induced PGD such as surface faulting and its effects on buried segmented pipelines. These effects include the nonlinear behavior due to the complexity of 3D joint geometry. Or, the excessive deformations of the pipeline materials, distortions in the pipeline cross-section, the contact interaction of surrounding soil with the pipeline, as well as the highly plastic behavior of some soils. The software also provides a wide range of material models that allow accurate simulation of ductile iron and highly nonlinear soil behavior. Additionally, Abaqus supports seamless integration with SolidWorks which allows detailed geometry to be imported for the pipeline and soil domain which is critical for the accuracy of the pipe joints. Therefore, it was concluded that these features make Abaqus/CAE a reliable and versatile tool for this research, and it was selected as the main tool of the study.

4.2. General Information About the Models and Their Preparation

The study involves six geometrical models originating from three cases of β (90° , 67.2° , and 45°) multiplied by two cases of γ (1 and 0.5). 12 distinct “soil block” parts were carefully drawn for geometric accuracy and carved out using the outer shape of the pipeline with computer aided drawing software SolidWorks to create a fixed and a moving soil block for each individual case. As shown in Figure 4.2, the fixed soil block (marked red) is held fixed in space while the moving soil block (marked beige) is laterally translated 1 m throughout all simulations. This motion hence simulates the perfectly lateral strike-slip faulting induced PGD on buried segmented pipelines for different cases.

As mentioned in previous chapters, there are many studies conducted on buried continuous pipeline systems. On the other hand, FEM models of buried segmented pipeline systems are scarce. In the approach of continuous pipeline studies, the lengths of the soil domain in longitudinal direction can range from 100 to 2000 meters due to

the properties and deformation nature of continuous pipeline systems. However, for segmented pipelines, such lengths for longitudinal domain could result in a computationally wasteful strategy due to their ability to rotate relatively more without having significant deformation and deform individually. Because of these reasons, the longitudinal length of the soil domain is chosen through multiple benchmark simulations and decided on using engineering judgment to capture the deformations on affected joints under the studied cases. 35 m and 29 m longitudinal length was chosen for the whole soil domain to include six pipes in total for cases of $\gamma=1$ and five pipes for $\gamma=0.5$ respectively. In order to avoid the influence of end conditions, the width and the total depth of the soil model was chosen to be 10 m and 7.5 m respectively. The burial depth (d) of 2.5 m was selected as an appropriate value for the pipeline as it reflects the typical in-situ applications of selected pipe class. Figure 4.2 shows the partial cross-sectional illustration of a 3D model created on Abaqus for the $\beta=90^\circ$ and $\gamma=1$ case.

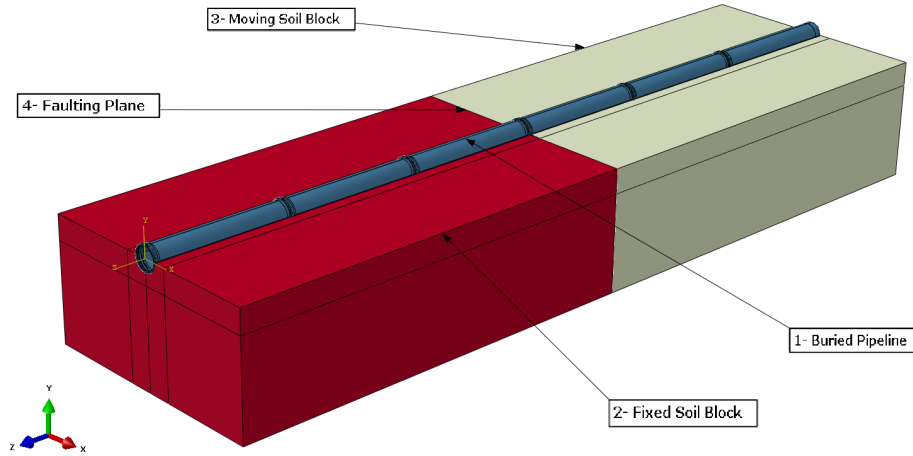


Figure 4.2. Visual demonstration of the 3D elements and the fault plane on partial cross-section of the model domain. Orthogonal fault crossing angle demonstrates how both fault displacement directions yield the same motion with respect to the pipeline and represents the neutral δ configuration.

The characteristic TYTON jointed pipe geometry shown in Figure 4.3 was based on the dimensions and specifications guided by the International Organization for Standardization (ISO) standard 2531:2009 and similarly was modeled in SolidWorks. A nominal diameter (DN) of 900 mm was selected as it was determined to be among the commonly used sizes. According to ISO-2531, the DI pipes that belong to this class has a total mass of approximately 1700 kilograms per working length of $L=6$ m. Other parameters such as the outer diameter (DE), length (L), thickness (e), and joint depth (P) were chosen according to the limits corresponding to the DN=900 as 945 mm, 6 m, 12.25 mm, and 200 mm respectively. For the adapted joint design, pullout failure is defined as the point where the relative axial displacement of the joint elements (ΔU_{rel}) exceeds the critical expansion limit (U_{crit}). The value of U_{crit} is obtained as 105 mm based on the formula

$$U_{\text{crit}} = L_{\text{eng}} - L_{\text{seal}} \quad (4.1)$$

where $L_{\text{eng}}=145$ mm is the engagement length and is defined by the length of the spigot that is inserted to the joint (beginning from the outer end of the seal) and $L_{\text{seal}}=40$ mm is the sealing length which is defined as the operating length of the gasket while maintaining full seal for the given joint design.

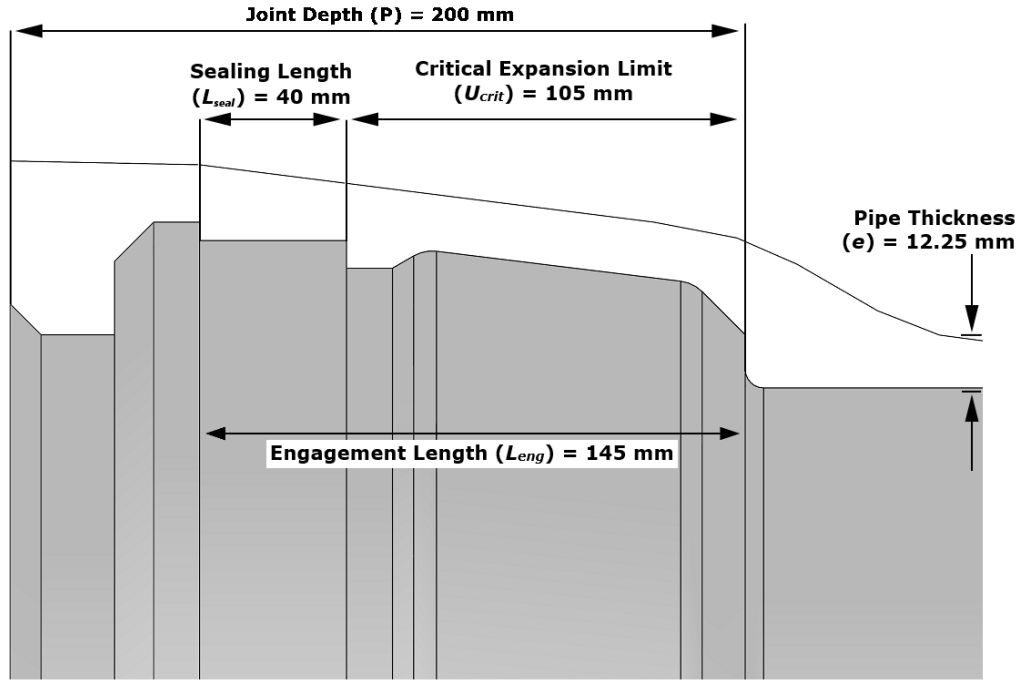


Figure 4.3. Cross-sectional view of the bell end of the TYTON joint illustrating the dimensions used in establishing a pullout criteria.

The joint design was selected to fall under the S-1 category defined by ISO 16134:2020, which classifies ductile iron pipes based on their extension and contraction performance. According to the standard, joints in this category must accommodate an axial extension of at least 1% of the segment length (L) before experiencing pullout failure. Given that the pipeline segment length is $L = 6$ m, the required axial displacement capability for S-1 classification corresponds to 60 mm. The critical expansion limit $U_{crit} = 105$ mm is greater than this value and meets the S-1 class requirements.

The pipeline was assembled by fully inserting the spigot end into the bell socket such that only 5 mm of longitudinal distance was left between the two segments. This setup allows the models to simulate the in-situ laying conditions while also leaving a small space to help initiate joint rotation without any obstacles in the way. Figure 4.4 shows the cross-sectional view of the pipeline joint assembly in numerical environment.

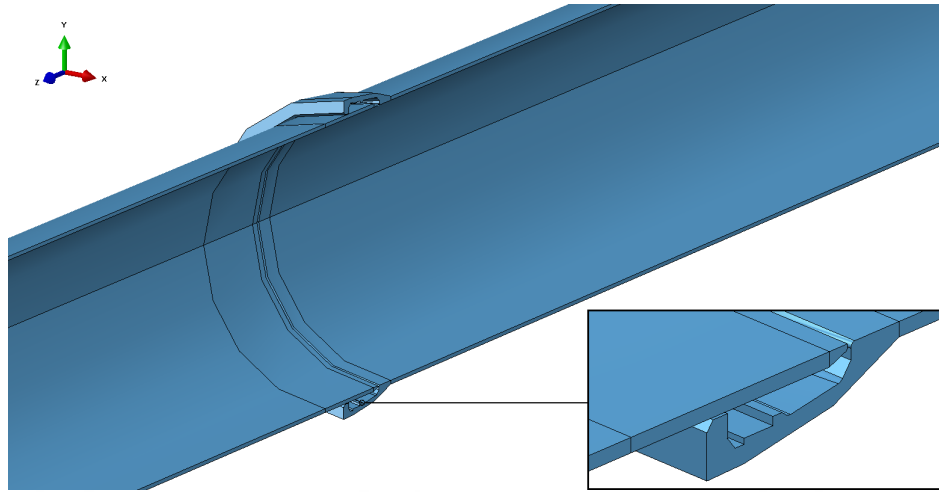


Figure 4.4. Longitudinal view of the cross-section of the pipeline joint assembly in Abaqus environment.

By examining the aspects mentioned above, the factors that the studied models aim to capture regarding the pipeline response include the effects of fault crossing angle (β), fault crossing position (γ), and fault crossing vector (δ) on the pipeline response; the effects of differences in soil stiffness and plasticity; the influence of joint geometry and the non-linear behavior caused by it; the effects of non-linear behavior of the soil-pipeline interaction on pipeline response; and the effects of non-linear behavior of the ductile iron (DI) pipe material, as well as the variety of surrounding soil materials, on pipeline response.

4.2.1. Benchmark Analyses and Optimization of The Models

Benchmark runs are critical steps in terms of optimizing the models to ensure the results in the main work are going to be reliable, accurate and obtained efficiently. They also serve as reference points for comparing simulation outcomes against each other to provide confidence in the model's ability to replicate real-world behavior. This is especially important as an overly simplistic modeling approach could greatly increase the required computational power or time to complete a simulation. Optimization involves adjusting the model parameters repeatedly until desired benchmark results

are reached in a minimum amount of time.

One of the important aspects of optimization procedure is determining an appropriate mesh size. It must be fine enough to capture the critical stresses, strains, and deformations in regions of high stress concentration, such as the pipeline joints or the soil-pipeline interface. On the other hand, the mesh be coarse enough in less critical regions such as those away from the pipeline, so that the solver won't spend excessive resources on that area and cause a slow simulation. An overly fine mesh can lead to excessive computational costs without a significant improvement in accuracy. Because of these reasons, a mesh convergence study during optimization phase is essential to find a balance between accuracy and computational efficiency.

Another critical consideration during optimization of the meshing is contact modeling. Optimizing the properties regarding contact ensures that the interactions between the soil-pipeline and pipe-pipe interface accurately reflect the physical behavior. The interaction between the “master” (the surface that is governing the contact) and the “slave” surface is highly nonlinear and requires adjustments in mesh sizes so that the size ratio is 2:1 respectively. This generally accepted principle is critical as lowering the ratio resulted in contact penetrations which eventually led to unrealistic displacements, stress distributions, followed by convergence issues in benchmark runs.

Benchmark simulations were also conducted to determine the dimensions of the soil domain to ensure that boundary locations are applied far enough from the pipeline and had no influence on the results. In literature, this distance is often chosen as 3–5 times the pipeline diameter in all directions from the pipeline's center. Therefore, it is concluded that the distance between the pipe centerline and the soil domain boundaries (other than burial depth) of 5 m is sufficient.

The two variation of $\pm\delta$ were also simulated beforehand to verify that the whole pipeline compressed as a system in $-\delta$ cases, and pullout motion is simulated for tension cases. To do this, $\beta=45^\circ$ case were utilized as it induced the highest amount

of longitudinal displacement to the system, meaning that observations could be made easier in terms of confirming that the pipeline behaved in accordance with its working principles.

Based on validation analyses, it has been observed that gasket elements require mesh sizes as small as 0.3 mm. Otherwise, numerical issues such as the gasket collapsing in on itself, failing to capture deformations, and significantly slowing down convergence rates are encountered. These issues can lead to dramatically increased simulation times. On top of that, a 2:1 mesh size ratio must be maintained for the pipe element to ensure proper functioning of the contact algorithm. Specifically, the pipe must be set as the master surface due to its stiffer nature, which requires the mesh size of approximately 0.6 mm to double the amount of the gasket. To maintain the same compatibility between the soil-pipeline interface, the soil mesh in the regions corresponding to the pipeline must also be refined according to the ratio of 2:1. This chain of mesh dependency causes the fine mesh to propagate from the gasket to pipe and from pipe throughout the whole soil domain on that axis. This approach is evaluated and considered unfeasible in benchmark analyses because of limitation of computational power. Moreover, since the validation analysis showed that the gasket does not significantly affect the results, and requires a very fine mesh to operate, it has been neglected in the main study.

Similar to the gasket, the internal operating pressure is also excluded in the main models using engineering judgment as it was observed that it only caused negligible amount of strain to the DI pipe in the validation analysis. This exclusion improved the efficiency of the simulation by reducing the required computational power and allowing the model to allocate resources to more critical aspects of the analysis (e.g., contact, fault displacement).

Lastly, Abaqus/Implicit solver has been experimented with to solve the simulation initially. However, the highly nonlinear nature of the problem led to cutbacks in simulation increments and extremely slow convergence after approximately 0.2 m of

fault displacement is applied. This highly nonlinear nature of the problem is primarily caused by excessive deformations on the soil and the pipe geometry. Implicit solvers are considered efficient for linear or moderately nonlinear problems. However, they can struggle to converge with such problems.

4.2.2. Material Definitions

In the main numerical models, the material properties of the pipeline and surrounding soil are chosen based on the relevant literature and standard practices. The same DI material properties (Wham and O'Rourke, 2015) from the validation analysis are utilized for the DI material in main numerical models. Also, two base soil models which represent soft clay and medium dense sand were tested for various faulting cases.

DI was chosen as the material for the pipes in the main parametric models. The primary reason is that their ductile behavior allows for accommodating moderate plastic deformation due to its strain hardening behavior before reaching the point of rupture, which is essential for examining the localized elastic as well as plastic strains on the joint system thoroughly. Also, DI was chosen due to its wide use in the industry for buried segmented pipeline applications, as well as its common association with the chosen pipe geometry. According to average of results from laboratory tensile coupon tests done by Wham and O'Rourke (2015), the material properties used for DI in conducted analyses have the 0.2% offset yield strength of 311 MPa and ultimate strength of 460 MPa with ultimate strain achieved at failure as 10.4%. The same data from tensile coupon tests from Figure 3.1 used in validation model are entered into Abaqus material module. The stress-strain data from Figure 3.1 was available only until the 325 MPa, beyond which the Ramberg-Osgood (1943) relationship

$$\varepsilon = \frac{\sigma}{E} + \alpha \left(\frac{\sigma}{E} \right)^n \quad (4.2)$$

was used to fit the missing data until the point of 460 MPa, where ε is the total strain, σ is the stress, E is the elastic modulus, and α, n are material-specific fitting constants.

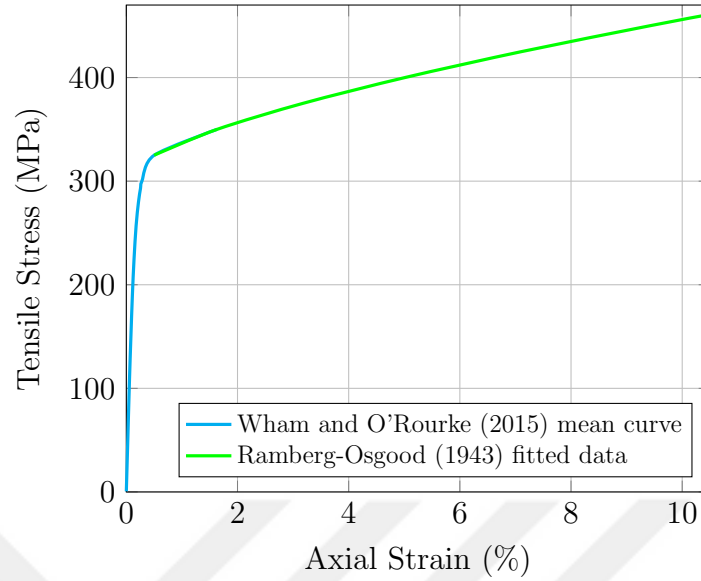


Figure 4.5. Stress-strain relationship for ductile iron pipe material. The figure combines experimental data from Wham and O'Rourke (2015) coupon tests (cyan curve) up to 0.5% strain, with Ramberg-Osgood (1943) model fitted data (green curve) for beyond 0.5% strain.

The materials of the surrounding soil deposit are chosen as both unsaturated soft clay and medium dense sand (MS) to create two parameters which demonstrate the effect of how different types of soil influence the pipeline's response. Like the DI model, both material models were set to show elastic behavior before reaching the yield state and show plastic behavior after that point. In addition to regular properties such as density and elasticity, Mohr-Coulomb material model was utilized in Abaqus to define their plastic behavior. The Mohr-Coulomb model calls for definition of cohesion and plastic behavior. The cohesion of selected soils was defined by entering the absolute plastic strains with corresponding yield stresses. Meanwhile, the geometrical aspect of plastic behavior was defined by entering an internal friction angle (ϕ) and a dilation angle (ψ).

The soft clay model was adapted from Demirci et al. (2018) and has a relatively low modulus of elasticity as well as low cohesion yield stress compared to medium dense sand. It was chosen as a parameter because of the similar reason mentioned for

DI. The highly plastic soil behavior under load allows the models to demonstrate the effects resulting from the highly nonlinear soil-pipeline interface interaction as well as its effect on the soil domain. Moreover, a high amount of plastic deformation of the surrounding soil is expected to allow better demonstration of the effects of faulting on the working principle of the segmented pipeline as a system.

Medium dense sand on the other hand, is considered significantly more stiff than soft clay. In this regard, it is expected from MS to be more resistant to deformation and allow the stresses on the pipeline to be localized more at the individual joint located at the fault interface. Also, more localized stress patterns can cause the joints to be subjected to higher forces, which could result in extreme deformations rather than a more distributed load leading to less deformation across the entire pipeline system. For this reason, it can demonstrate the independent behavior of the joints better. The medium dense sand model was also adapted from Demirci et al. (2018), of which its properties as well as those of soft clay and DI are provided in Table 4.2. Note that a non-zero cohesion value was chosen (1 kPa) for sand model in order to avoid crashes in the simulations and imitate 0 cohesion of sand particles.

Table 4.2. Summary of information entered during the model assembly.

Component	Property	Unit	Value
Ductile Iron Pipe	Elastic Modulus, E_p	GPa	186
	Yield Strength, σ_y	MPa	311
	Mass Density, ρ_p	kg/m ³	7100
	Poisson's Ratio, ν_p	—	0.29
	Segment Length, L	m	6
	Nominal Diameter, DN	mm	900
	Wall Thickness, e	mm	12.25
	Burial Depth to pipe centerline, d	m	2.5
Soft Clay	Elastic Modulus, E_{sc}	MPa	25
	Mass Density, ρ_{sc}	kg/m ³	1650
	Poisson's Ratio, ν_{sc}	—	0.3
	Friction Angle, ϕ_{sc}	°	0
	Dilation Angle, ψ_{sc}	°	0
	Cohesion Yield Stress, c_{sc}	kPa	50
Medium Dense Sand	Elastic Modulus, E_{ms}	MPa	20
	Mass Density, ρ_{ms}	kg/m ³	1800
	Poisson's Ratio, ν_{ms}	—	0.35
	Friction Angle, ϕ_{ms}	°	32
	Dilation Angle, ψ_{ms}	°	1
	Cohesion Yield Stress, c_{ms}	kPa	1

The two soil models mentioned above are selected to yield significantly different results from the commonly used Mohr-Coulomb failure criterion equation

$$\tau = c + \sigma_n \tan(\phi) \quad (4.3)$$

that outputs the shear strength of soils (τ), where c is the interparticle bonding strength (cohesion) of the soil, σ_n is the normal stress acting on the soil failure plane, and ϕ is the internal friction angle of the soil (resistance to shear due to friction). This method

is adapted because it is expected from shear strength (τ) of the soil to govern the effect of change in soil stiffness on pipeline response. Moreover, choosing multiple soil models allows comparison of different soil behavior's influence on pipeline deformation.

4.2.3. Contact Definitions

General contact was used to represent all contact interactions in the models because it includes every other type of geometric interaction (e.g., surface-to-edge contact) possible unlike surface-to-surface contact. The general contact solver is considered significantly more robust but comes with higher computational costs.

The friction coefficient (μ) of 0.3 and 0.6 was assigned to the soil-pipeline interface to model tangential behavior of the soft clay and medium dense sand respectively. This difference in μ is due to softer clay having lower shear strength compared to sand, which makes its friction mechanism based more on adhesion. Sand on the other hand, has more rigid and granular particles that interlock with the adjacent surfaces. DI pipes in this regard are highly prone to this type of friction due to their rough surface finish. Thus, the friction coefficient between the sand and DI must be higher than that of clay and DI. Moreover, the values of μ are approximated from multiple reference papers (Li et al., 2024; Demirci et al., 2018), which include interactions of different soils with rougher cast iron pipes and smoother steel pipes.

Due to being significantly stiffer than the soil, DI pipes were selected as the master surfaces governing the interaction while surfaces of the soil are selected as slave. And since the DI material properties are the same as those of validation analyses, the μ of 0.65 is opted again to model the friction between the pipe segments at joints. This value is chosen to be high enough to represent real life conditions in which the surface of DI pipes is relatively coarser to that of steel pipes, which are generally modeled with $\mu=0.5$. The normal behavior of contact for the whole model is defined using “hard contact”.

4.2.4. Mesh Configuration

A mesh is a numerical representation of a problem used in FEA to discretize complex structures or regions into smaller, manageable elements to solve that problem for each of the elements individually. These elements can take various shapes, such as hexahedral, quadrilaterals, tetrahedral, or wedge depending on the complexity of the geometric shape of the model.

As previously explained, the mesh must be adjusted to be neither too fine nor too coarse so that it does not slow down the computational process. Meanwhile, it must still be capable of accurately handling computationally expensive tasks such as element calculations. Additionally, as will be discussed shortly, there are other critical factors influenced by the mesh size that must also be considered as well. Addressing these aspects requires extensive study and optimization. For these reasons, the mesh size of the pipes in the studied models has been adjusted to find a balance between an overly coarse and an excessively fine mesh.

For every model, both the pipeline and the soil are modeled using hexahedral, 8-node linear brick, reduced integration with hourglass control (C3D8RH) elements. These elements are generally considered ideal for efficiently capturing large plastic deformations of soil and the pipes while maintaining computational costs low.

In radial direction, the pipe as well as its surrounding thin layer of soil mesh are discretized with mesh size of 5 cm along their circumferences. This is done to ensure that both domains match with one another and surface intersections are avoided.

Figure 3.1 illustrates the mesh configuration of the pipe elements. In longitudinal direction, the segment portion of the pipe which is 25 cm away from the joint ends is meshed with size of 30 cm. Joints on the other hand, are meshed with 5 cm longitudinally because of the requirement of their more detailed complex geometry. Longitudinal mesh sizes of the soil are chosen as 15 cm (half of that of pipe's) to obey

the rule of 2:1 size ratio for the stability in contact surfaces.

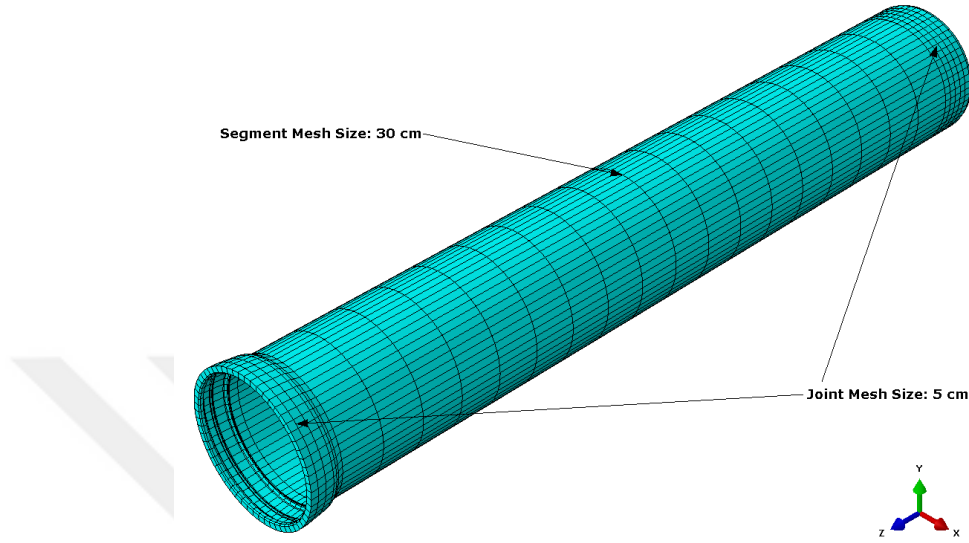


Figure 4.6. Finite element mesh configuration of the 6-meter pipe specimen used in the simulations.

The mesh of the soil blocks is tuned such that the farther the soil is from the pipe, the mesh size gets coarser. This approach is done to optimize computational costs without sacrificing accuracy in critical areas. Fine meshes are required near the pipe to capture the detailed stress and deformation interactions between the pipe and the surrounding soil. However, as the distance from the pipe increases, the gradients of stress and strain are expected to diminish, meaning less detail is required to represent the soil's behavior accurately. Moreover, a thin layer of soil with 5 cm thickness for mesh is used for the initial 5 cm that is surrounding the circumference of the pipe. Then, the mesh size increases to 15 cm and continues until the offset of 1 m from the pipe centerline in vertical and horizontal axes. Finally, after 1 m, a mesh size of 0.5 m is used until the boundaries of the domain where no significant element calculations are expected in results. Figure 4.7 shows the mesh configuration of the soil blocks in 3D model.

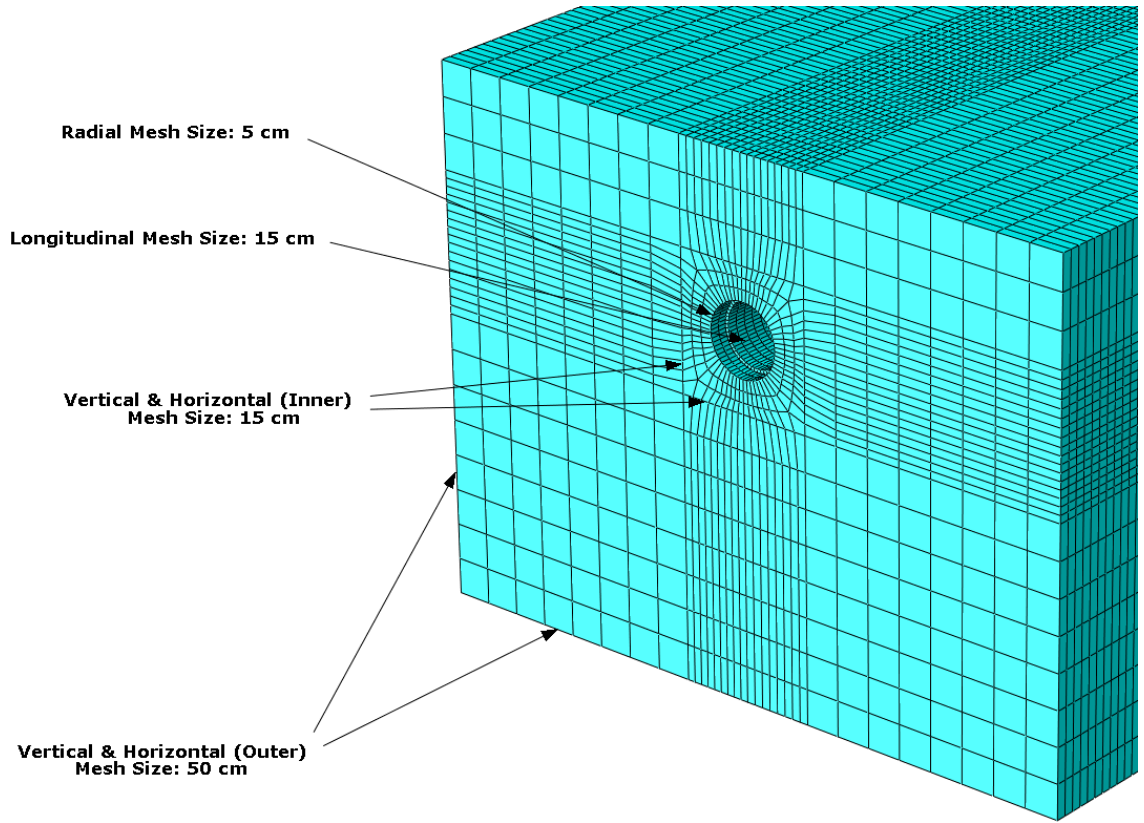


Figure 4.7. Finite element mesh configuration of the soil domain used in the simulations.

4.2.5. Longitudinal End Conditions of the Pipeline

As mentioned before, buried segmented pipelines can extend several kilometers long, and the boundaries of the numerical models in this study simulate only a certain portion. However, it is expected that the joints of the remaining missing pipes at the model's boundaries could contribute to the response in specific scenarios where δ is in the negative direction. In the absence of a solution that longitudinally constrains the joints on both ends, the pipeline could experience no compression, and it may slide out of the soil domain as the faulting motion occurs. Because of this reason, the continuity of the whole pipeline must also be defined by simulating the end conditions in numerical environment.

In such studies where continuous pipelines are of subject, this effect can be easily modeled by following the same boundary conditions of the soil edges at both ends of the model. However, a different solution must be adapted for segmented pipelines as missing joints located at the edges of their numerical models behave differently than the end of a continuous pipeline.

To address this, special end conditions are defined for the two pipes at both ends of the parametric models. The circumferential nodes at the ends of these two pipes are kinematically coupled to two artificial nodes located at the pipe centerline, similar to the flexible joint used in preliminary validation study. Then, only the longitudinal motion of these two nodes is governed by the same boundary conditions used for the soil at the longitudinal boundaries of the model. Meaning that any other transition or rotation of the nodes surrounding the pipes at the far most points of the pipeline is still unconstrained. This specially designed end condition for $-\delta$ cases allowed simulation of the effect of joints that are excluded in the model's domain and the continuity of the pipeline.

4.3. Critical Considerations

4.3.1. Abaqus/Explicit solver

In FEM analyses, node calculations (acceleration, velocity, displacement) are considered “cheap” with current average computational resources. However, element calculations, such as strains, stresses, and internal forces, are relatively “expensive”. Explicit solvers are better suited for types of analyses that aim to obtain element calculations. They can handle large deformations, highly nonlinear behavior, and abrupt changes in loading or boundary conditions more robustly.

The main advantage of Abaqus/Explicit over the standard solver is that it doesn't require inverting the stiffness matrix during the solve. This is because it utilizes diagonal lumped mass matrix and therefore doesn't require multiple Newton-Raphson

minimization increments (or similar iterative matrix solver) to find a solution. Hence, it does not need to check if the equations converge for every step of the simulation and eliminates the divergence in solutions.

Instead, explicit solvers take thousands of minuscule steps to solve for the next step forward in time. Which makes them much more beneficial for contact and large mesh distortion events such as earthquake-induced fault displacement studies. For this reason, Abaqus/Explicit solver is adopted in this study to ensure that the simulations are stable and accurate even under extreme loading conditions.

4.3.2. Quasi-Static Assumptions in Abaqus/Explicit Solver

As previously mentioned, the response of buried pipeline systems to ground deformation is not significantly affected by dynamic effects such as inertia. This is due to lack of space for the pipelines to gain significant acceleration or velocity relative to the soil domain they are placed in.

Quasi-static analysis is a simulation method used to model scenarios where the effects of inertia and damping are negligible due to the slow application of loads or displacements. This approach assumes that the system remains in static equilibrium throughout the process. It may save a significant amount of computing power depending on the case, or such as this case, can be utilized to eliminate dynamic effects and resulting unrealistic stress accumulations, as well as excessive deformations caused by inertia.

Quasi-static analyses are readily available for the user to turn on in implicit solvers but require careful setup to ensure its criteria are met in explicit simulations. To ensure that the response of the pipeline system is obtained quasi-statically, the ratio of the total kinetic energy that is accumulated (ALLKE) to the total internal energy (ALLIE) in the system is checked throughout the analyses. The conventional method involves that this ratio should be kept around the threshold of 0.05 to ensure that

the overall results are not being influenced by inertial effects. It can be invalidated. Quasi-static assumptions can be invalidated by both increasing the rate of the analysis (by decreasing total step time) or by excessive use of mass scaling. For these reasons, the kinetic energy accumulation must be reduced throughout the analyses. And the simulation period must be carefully selected just long enough to allow a quasi-static simulation and short enough to not waste computational resources.

4.3.3. Stable Time Increment in Abaqus/Explicit Solver

In explicit finite element analyses, the global stable time increment is a parameter that determines the maximum allowable time step size for the simulation to maintain numerical stability. It is governed by individual minimum stable time increment of all the elements in the mesh, which is calculated based on each element's material properties (density and stiffness) and mesh length. For example, even if 99.99% of the elements in a mesh domain have a stable time increment of 1×10^{-3} seconds, a single element with a stable time increment of 1×10^{-9} seconds would still determine the global time increment for the entire model.

To modify the global stable time increment, three theoretical options exist: adjusting the element stiffness, increasing the material density, or increasing the characteristic length of each individual mesh. However, changing stiffness is generally avoided, as it would fundamentally alter the physical behavior of the model. Therefore, the practical methods to increase the global stable time increment are either to increase the material density (by using mass scaling) or to increase the characteristic length of the smallest mesh elements.

In the established models, the smallest mesh elements with governing size belong to the pipe specimens (specifically their joints) due to their detailed geometry and requirement finer mesh for capturing the more expensive highly nonlinear element calculations. This is another part of the model which needs careful consideration as the mesh sizes cannot be too coarse. Meanwhile, since joint mesh sizes govern the

minimum stable time increment of the simulation, cannot be created overly fine as well. Because of this reason, the global stable time increment was controlled using mass scaling feature of Abaqus/Explicit.

4.3.4. Mass Scaling in Abaqus/Explicit Solver

Mass scaling is a method used in explicit FEM analyses to artificially increase the density of problematic elements thereby increasing the global stable time increment. By adding a small amount of additional mass to problematic elements with very small stable time increments, the time step can be increased. As a result, it allows the computational cost to reduce while maintaining the overall kinematics of the system for obtaining still accurate results.

The mass scaling is set to apply to the whole model at the beginning of the simulation and automatically update the scaling factor to meet the target stable time increment of 8.75×10^{-6} seconds rather than blanketing a fixed density multiplication factor across the whole mesh. This value was optimized through benchmark analyses to ensure that the total mass increase in the simulation remained below 5% to maintain the physical accuracy of the models.

4.4. Analysis Procedure

The analyses are conducted in two steps, the first step is for the gravity loads to apply to the whole model, and latter step is for the simulation of the faulting motion on the moving block. Normally, with type of steps such as the first one, often “Static-General” solver in Abaqus/Standard is utilized. It is particularly suitable for such simulations involving gradual loading and when dynamic effects such as inertia and damping are completely negligible. Simulating problems that are suitable for this solver with Static-General option has many computational benefits as it can solve this step in just few minutes. It provides various stabilization options that are easy to set up for better convergence. It also supports adaptive time stepping for automatically

adjusting time increments based on model stability and nonlinearity.

The second step involves dynamic events and excessive deformations that are highly nonlinear in many aspects, and therefore require simulating using an explicit solver. A complexity in this regard is that switching particularly from the static-general solver to dynamic-explicit solver between steps requires an “import analysis” between the two Abaqus versions (Abaqus/Standard - Abaqus/Explicit). An import analysis is a highly time consuming and difficult process which needs the model to be created from the start. Every single aspect of the model that is being imported needs to be defined again. On top of that, the resulting node and element calculations at the end of the first step must be carefully carried over and applied to the new model with an already deformed shape. And since this study is parametric, the import analysis leads to the requirement of creating all 20 models from the start and going through the entire process for each individual model. This process not only doubles the number of models which require careful attention but also adds additional considerations which are relatively more difficult to assess. For this reason, using a static-general solver for the Step-1, and eventually doing series of import analyses is deemed unfeasible. Thus, both the Step-1 and Step-2 of all models are simulated in the explicit solver.

4.4.1. Numerical Application of the Gravity Load

In Step-1, the self-weight of the soil blocks as well as 1700 kg for each of ductile iron (DI) pipes are applied through gravity load of 9.81 m/s^2 . In this phase, all lateral sides and the bottom boundary surface movement is restricted on their individual normal direction using displacement boundary conditions. The period of this step is chosen as 0.5 seconds to allow the weight of the pipes to fully propagate to the soil domain. In the first step (referred to as the “gravity step” from now on), quasi-static assumptions are not as significant as the second step (referred to as “faulting step” from now on). This is because the gravity step does not involve dynamic loads or rapid displacements that could generate significant inertial effects. Meaning that the first stage of the simulation can be run without considering the ALLKE/ALLIE ratio.

4.4.2. Numerical Application of the Fault Displacement

Step-2 involves the lateral strike-slip fault movement. The fault movement is applied in this phase by changing the displacement boundary conditions of the moving soil block to yield 1 m of displacement in the desired fault crossing angle's β direction. For example, 0.7071 m of displacement is applied in both lateral axes for the $\beta=45^\circ$ case to obtain 1 m in result using the concept of vector decomposition. Meanwhile, the fixed soil block's original boundary conditions are kept fixed in place to simulate the stationary part of the PGD induced soil. The fault displacement is applied quasi-statically by keeping ALLKE/ALLIE ratio around 5%, with a linear amplitude distributed to 20 seconds. The series of analyses are run until either the 1 m of fault displacement is achieved or the ultimate strength of the DI material (460 MPa) is reached.

5. RESULTS

The 20 scenarios previously shown in Table 4.1 were numerically analyzed for all combinations of β , γ , δ , and two soil models to simulate and investigate response of a segmented ductile iron pipeline subjected to perfectly horizontal strike-slip surface faulting. The moving soil block is gradually translated to apply 1 m of displacement in total of 20 seconds, corresponding to 0.05 m/s of displacement.

The following chapter provides the observations on obtained results using relevant tables figures etc. Note that, after an initial section of the general overview, the results are provided in two main sections according to two evident cases of segmented pipeline failure modes under varying fault crossing vectors. The PGD cases including various mechanisms of surface faulting resulted in fundamental differences in pipeline kinematics, stress localizations, and consecutive failure modes. It was observed that both in tension inducing $+\delta$ and neutral δ ($\beta = 90^\circ$) cases, the pipeline failure mode was governed by the pullout failure. Whereas on $-\delta$ cases, material failure caused by exceedance of ultimate stress of DI caused the joint crushing failure mode. Furthermore, the two sections were created in order for easier data processing and to avoid the blending of different trends. A part of the results presented in this chapter was recorded for every 5 cm of fault displacement and linearly interpolated due to hardware limitations, whereas others were available for every minuscule time increment. Throughout this chapter, $\beta = 67^\circ$ in figures refers to $\beta = 67.5^\circ$ cases for simplicity in labeling.

5.1. Overview of General Observations and Trends

For cases where γ is in positive or neutral direction, the amount of stress exerted on the pipeline by soil increased with the increasing β value. The stiffer soil, represented by the medium dense sand deposit with higher shear strength, applied greater stress and rotation to the pipeline's body. However, in all cases of $+\delta$, these stresses normal

to the pipeline body remained below the material yield threshold of the pipeline. In scenarios where $\beta = 45^\circ$, higher longitudinal displacements allowed easier visualization of the different amounts of friction forces exerted on the outer body of the pipeline by the two soil models. In this regard, medium dense sand model exerted relatively higher grip to the outer body of the segments compared to soft clay model. However, there wasn't any significant evidence of effects of friction influencing the pipeline response.

For cases where pullout failure occurred, maximum number of joints experienced the failure at the same time was limited with three. For cases where material yield threshold is achieved, the plastic deformations occurred mostly at the joints, whereas body of the pipes remained in the elastic strain region. The stresses in these cases are distributed to the whole pipeline domain. Depending on the case, plastic joint deformations are observed on multiple joints before pipeline rupture has occurred. These deformations mainly occurred on spigot and rarely on bell elements.

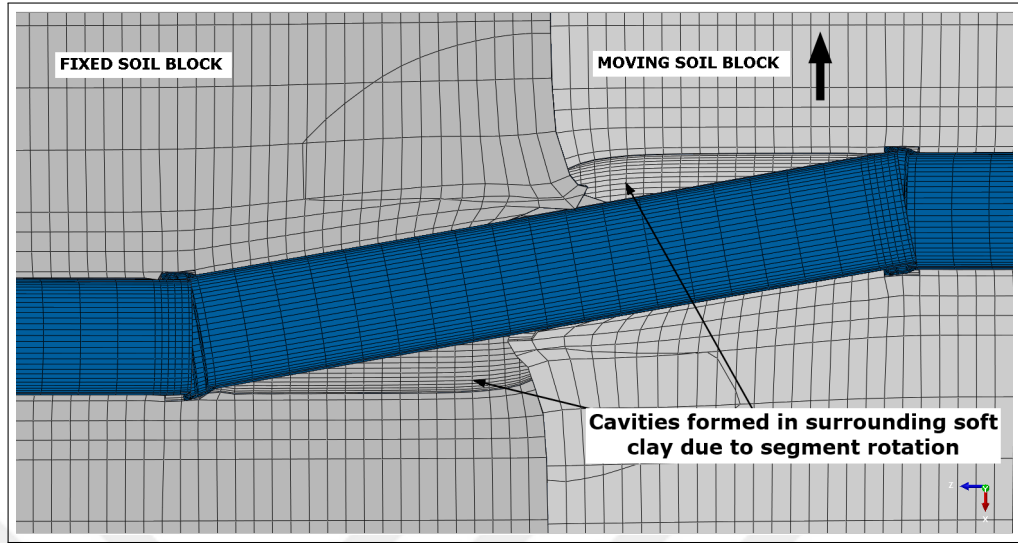
Table 5.1 shows the summary regarding the pipeline failure types corresponding to all cases studied in the thesis. Two cases with the same $\beta = 90^\circ$ and $\gamma = 1$ did not reach the either pullout or material failure criteria defined in 4.

Table 5.1. Summary of pipeline failure analysis across different fault crossing parameters and soil conditions. Table indicates pullout failure by the standard criterion.

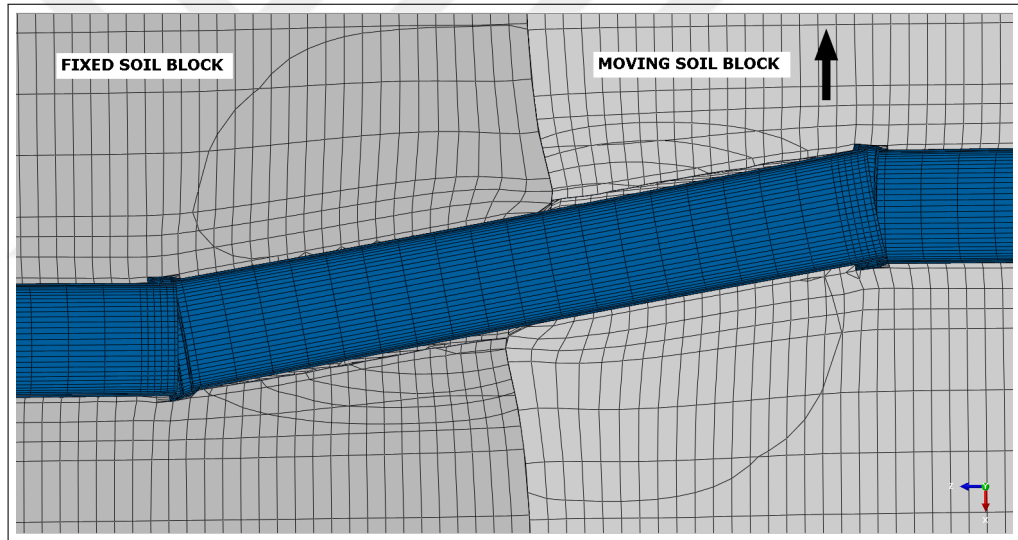
Case No.	Fault Angle (β)	Fault Position (γ)	Fault Vector (δ)	Type of Soil Deposit	Type of Pipeline Failure
1	90°	0.5	n/a	S. Clay	Pullout F.
2	90°	1	n/a	S. Clay	NO FAILURE
3	90°	0.5	n/a	M.D. Sand	Pullout F.
4	90°	1	n/a	M.D. Sand	NO FAILURE
5	67.5°	0.5	+	S. Clay	Pullout F.
6	67.5°	1	+	S. Clay	Pullout F.
7	67.5°	0.5	+	M.D. Sand	Pullout F.
8	67.5°	1	+	M.D. Sand	Pullout F.
9	67.5°	0.5	—	S. Clay	Joint Crush. F.
10	67.5°	1	—	S. Clay	Joint Crush. F.
11	67.5°	0.5	—	M.D. Sand	Joint Crush. F.
12	67.5°	1	—	M.D. Sand	Joint Crush. F.
13	45°	0.5	+	S. Clay	Pullout F.
14	45°	1	+	S. Clay	Pullout F.
15	45°	0.5	+	M.D. Sand	Pullout F.
16	45°	1	+	M.D. Sand	Pullout F.
17	45°	0.5	—	S. Clay	Joint Crush. F.
18	45°	1	—	S. Clay	Joint Crush. F.
19	45°	0.5	—	M.D. Sand	Joint Crush. F.
20	45°	1	—	M.D. Sand	Joint Crush. F.

It was observed that after the initial gravity step, the stress contours on the soil domain were influenced by the shape of the segmented pipeline (including enlarged portions of the pipeline) projecting on the soil domain. This indicates that the pipeline's weight was successfully transferred to the surrounding soil.

The effects of the soft clay model were evident, as its relatively low stiffness and shear strength made it highly prone to deformation. The field output in Figure 5.1a showed that the cavities continued to form around the pipeline in scenarios where soft clay was used, which indicates its inability to provide a strong confinement. This is attributed to the material's low friction angle and lack of dilation, which caused the low stiffness. In contrast, this effect was not present in medium dense sand on Figure 5.1b, where the higher stiffness allowed the soil to mobilize shear resistance, which only allowed negligible amount of cavity formation around the pipeline.



(a)



(b)

Figure 5.1. Soil deformation comparison for $\beta = 90^\circ$ and $\gamma = 0.5$ cases. a) illustrates the cavities in the surrounding soft clay due to the segment's rotation, while b) shows the deformation pattern of medium dense sand with no cavity formation.

In medium dense sand models, heaving of the soil surface was observed with maximum height of nearly 11 cm. This height was observed at its highest at the $\beta = 90^\circ$ value, which can be attributed to increasing amount of segment rotation and consecutively higher stresses exerted on the surrounding soil. This soil response is attributed to the higher stiffness and lower compressibility of the sand compared to

soft clay. As the pipeline imposed displacement onto the surrounding soil, the relatively stiffer sand transferred the load more efficiently and caused upward deformation near the ground surface. The heaving occurred along the surface trace of the fault interface and was accompanied by adjacent ground subsidence. The subsidence observed in MS cases was caused by cavities left behind as the pipe segment rotated while leaving cavities in soil domain in its behind. These voids were subsequently filled by the surrounding sand. No visible change at the surface was observed in soft clay models as they accommodated stresses through plasticity without any noticeable uplift.

As mentioned before in Chapter 4; to assume a quasi-static simulation in explicit solvers, the internal energy and kinetic energy of the models must be monitored. And the commonly adapted method is to make sure that the ALLIE/ALLKE ratio is around 5% at the end of the simulation. The Figure 5.2 below shows the plots of these values, and the total energy balanced in the system (ETOTAL) for a single case as an example. Note that the ALLIE values fluctuate due to shortage of simulation period (20 seconds), which was chosen based on the minimum requirements to ensure the simulations are quasi-static. This decision was made due to time constraints as the parametric study concerned with 20 models, each requiring approximately two days to complete.

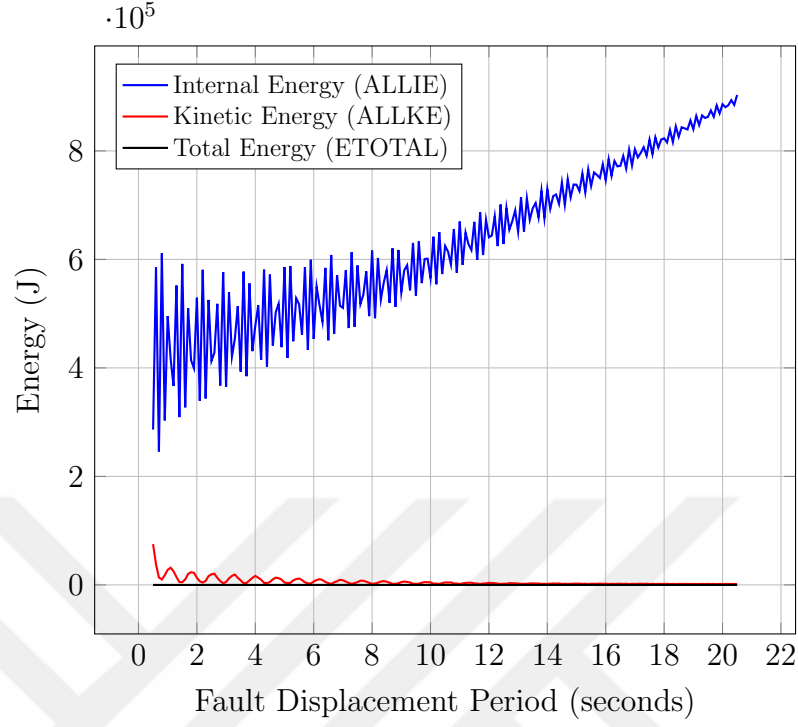


Figure 5.2. Energy content for a single model, showing variation of ALLKE, ALLIE and ETOTAL with time.

Figure 5.2 shows that towards the end of the simulation, ALLKE values diminish, and the required ALLIE/ALLKE ratio is met. Meaning that the simulations are ran quasi-statically, and inertial effects do not dominate the analysis. In addition, a side note could be that introducing 1 m of fault displacement in total of 20 seconds could be considered realistic depending on the case. Furthermore, in cases where there is minor inertia present in the model, it is neglected.

It can also be observed that the oscillations in ALLIE gradually stabilize as the simulation period gains progress. This behavior can be attributed to the initial dynamic adjustments of the system where sudden loading conditions and contact interactions at the beginning of the simulation induce transient energy fluctuations. As the simulation advances, the system reaches a more stable state and reduces the magnitude of oscillations.

5.2. Perpendicular and Positive Fault Crossing Vectors

In this section, various details are provided for four cases of perpendicular γ and eight cases of tension-inducing ($+\gamma$) scenarios to allow clarity in understanding the pipeline response in these cases. In this case group, either pullout occurs or no failure is obtained in results. A single joint representing the weakest link per simulation (including cases where multiple pullout behavior occurred) was selected and analyzed for results. This is because one critical joint in every model experienced relative axial displacement between its elements faster than others. Hence, it ultimately governed the overall failure of the pipeline.

The results for ΔU_{rel} are obtained by extracting the axial displacement of elements that belong to the said joint from each other, and taking the absolute value. The influence of rotation on axial separation is accounted for by defining an individual coordinate system to each of the involved segments and evaluating separation on their own longitudinal axes. A complication that would be normally encountered while measuring the ΔU_{rel} is that the possibility of pipeline rotating in any of the three directions. However, in the set of analyses included in this thesis, a significant advantage is that all the considered scenarios simulate a perfectly horizontal fault movement. Meaning that the axial separation of joint elements can be extracted solely from the pipe springline, which otherwise would be done from the centerline to account for the vertical separation as well. These approaches are evaluated as effective to use in this thesis for comparing the effects of different parameters and capturing the essential failure characteristics without unnecessarily complicating the data extraction.

In addition, an alternative criterion for defining the pullout failure was adapted to obtain less conservative results that capture significant changes in pipeline performance. Moreover, in addition to the “standard” pullout criteria ($U_{crit} = L_{eng} - L_{seal} = 105$ mm) explained in Section 4.2, the gasket sealing length (L_{seal}) was assumed to be 20 mm instead of the more conservative 40 mm, resulting in a critical displacement threshold U_{crit} of 125 mm. This less conservative criterion represents the assumption

that even if the axial separation of joint element reach the half of the gasket sealing length (L_{seal}), the remaining half of its seal zone could still maintain an effective seal. Moreover, the pipeline may continue to fully function as long as the displacement does not exceed the midpoint of the sealing region, where the sealing performance should be hypothetically at its highest due to bulb shape of TYTON gaskets. Under the “less conservative” assumption, the normalized axial displacement remained below the failure threshold over a broader range of fault displacements.

5.2.1. Kinematic Response of the pipeline

This subsection mainly concerns the kinematic response because there wasn't any plastic deformations recorded near the gasket seal zone. And the pipeline components could not reach their ultimate stress in any of the tension inducing ($+\gamma$) and perpendicular (γ) cases. Instead, all of these scenarios resulted in either pullout failure or no failure in any extent. Furthermore, axial separation as well as the rotation of the pipe segments was the dominant mechanism to be investigated.

The Figure 5.3 shows the points of failure for the two failure thresholds and relative distance between accommodated fault displacements. For cases where $\beta = 45^\circ$, pullout failure occurred around the 33-57 cm fault displacement range for the two failure thresholds while these ranges are relatively higher in $\beta = 67.5^\circ$ and $\beta = 90^\circ$ cases. Meaning that the relative fault displacement accommodation between failures points are increasing as β angle gets closer to 90° . This suggests that the β angle gets more perpendicular, the influence of U_{crit} on the pipeline performance for pullout failure increases.

The figure also shows a clear influence of the adopted pullout failure criteria on the interpretation of pipeline performance. In the standard failure criterion, the pipeline reaches pullout at a lower fault displacement, while in the less conservative criterion, it tolerates greater fault movement before failure. Under the standard failure threshold, both of $\gamma = 0.5$ cases suffered pullout failure with soft clay case at around

75 cm and medium dense sand case at 80 cm regardless of the type of soil deposit. Whereas in less conservative failure threshold, it occurred at around 93 cm for soft clay case and 97 cm for the medium dense sand case.

This adjustment in failure criteria resulted in a total difference of approximately 20 cm additional fault displacement before failure for $B=90^\circ$ cases. This demonstrates how the choice of failure criterion can significantly affect the pullout performance of pipelines.

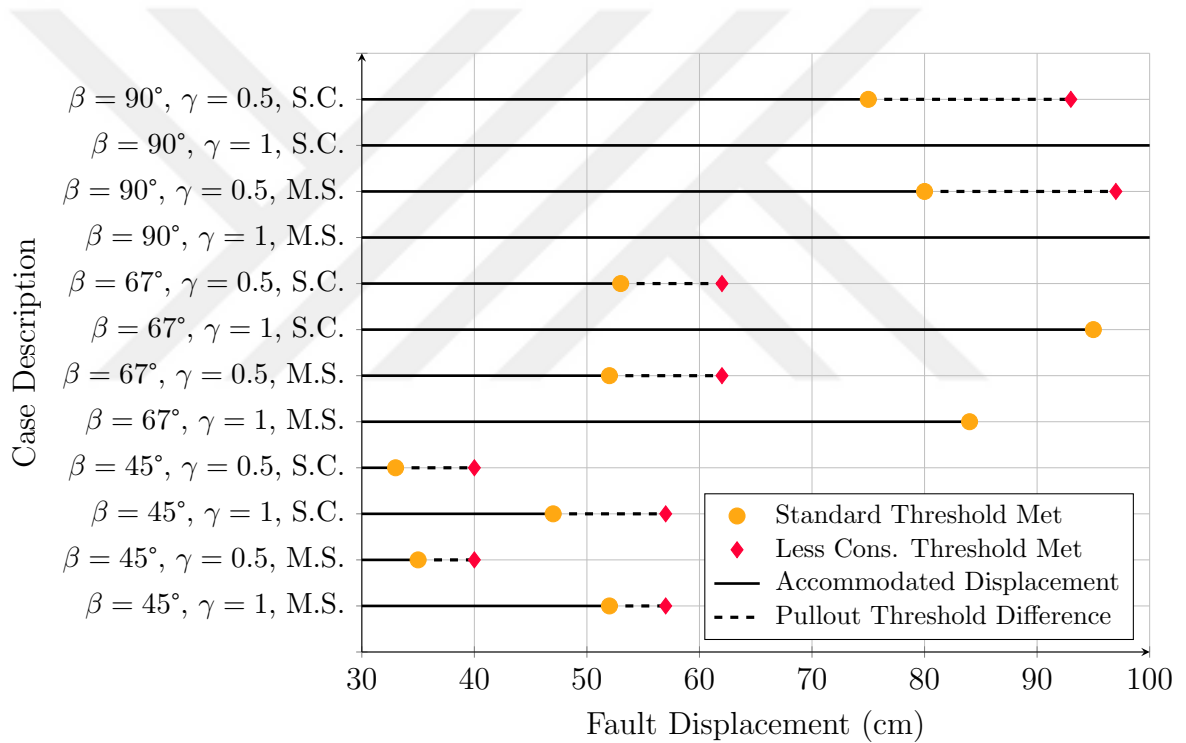


Figure 5.3. Distribution of failure points across different cases. Orange circles represent the standard pullout threshold, while red diamonds represent the less conservative pullout threshold.

The Figure 5.4 shows that the obtained relative joint rotation values at failure decreased as the angle of β gets narrower. Meaning that the joints failing early could not rotate as much as other cases. These trends indicates the effect of narrower β angle exerting relatively more longitudinal displacement and consecutive axial separation to the pipeline. On the other hand, perpendicular β applies greater rotation to the

involved segments.

Another effect can be observed at both when $\beta = 45^\circ$ and $\beta = 67.5^\circ$. Relatively higher joint rotations were recorded in cases where $\gamma = 0.5$ compared to $\gamma = 1$ cases. Meaning that higher joint rotations are exerted on the pipeline if the fault is crossing from the middle of the pipe segment.

In addition, when all the other parameters are individually kept the same, medium dense sand models consistently induce higher rotation in the pipeline joints than soft clay models. This can be attributed to the stiffer nature of medium dense sand likely restricting soil deformation and lateral movement of the pipe. And consecutively causing the pipeline to rotate more at the joint before reaching failure.

Main takeaway from this figure is that, by looking at the consistent trends in magnitude of differences in rotation at failure, the level of influences of different parameters to these values can be listed. Among the examined parameters, fault crossing angle (β) shows the most significant influence, followed by fault crossing position (γ), while the difference in two soil models shows the least effect on rotation at failure for cases of pullout failure.

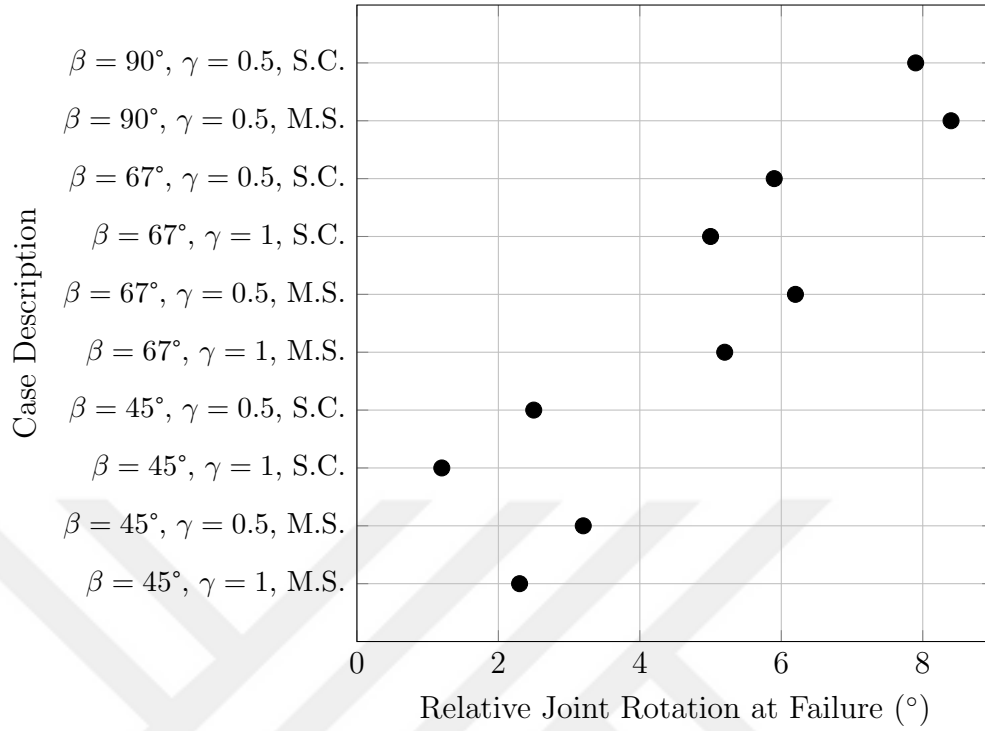


Figure 5.4. Scatter plot showing the relative joint rotation at failure for different fault configurations for the standard pullout failure criterion (cases that did not reach failure were excluded).

It is worth mentioning that the typical rotational limits specified by manufacturers for TYTON joints are generally around 5–6 degrees. However, the simulation results demonstrated that pullout failure can occur at significantly higher rotation values. In one of the cases, a joint accommodated up to 8.4° of relative rotation before failure was triggered. This may be attributed to the conservative nature of manufacturer-provided limits, which likely include safety factors or represent thresholds beyond which joint rotation may begin to initiate pullout rather than cause immediate failure. Additionally, it is plausible that even greater rotational capacity could be observed with slight adjustments to the fault geometry parameters. This indicates a potentially wider performance margin under specific loading conditions.

5.2.2. Interlocking Segments

It was observed in cases where δ is neutral or in positive direction, and fault crosses the pipeline from the middle of the joint ($\gamma = 1$), the bell and spigot elements that belong to that joint in between the two fault planes lock onto each other. These interlocking two segments act as one pipeline segment (with double the size) and rotate together. Notably, the joint interlocking phenomenon was observed in every scenario defined above where the fault crossed the pipeline from its joint, regardless of changes in β or the surrounding soil deposit. It was also observed that interlocking pipe elements caused the stresses to be localized mainly on the bell entry involved in each simulation it occurred. These stresses did not exceed the yield point of the material. Ultimately, the affected joint did not experience pullout failure.

A critical observation is that the only two cases which the pipeline did not fail under the standard failure threshold ($U_{crit}=105$) are the those where joint interlocking is observed. Furthermore, in the two additional cases where the less conservative pullout threshold ($U_{crit}=125$ mm) was not reached, interlocking phenomena was again present.

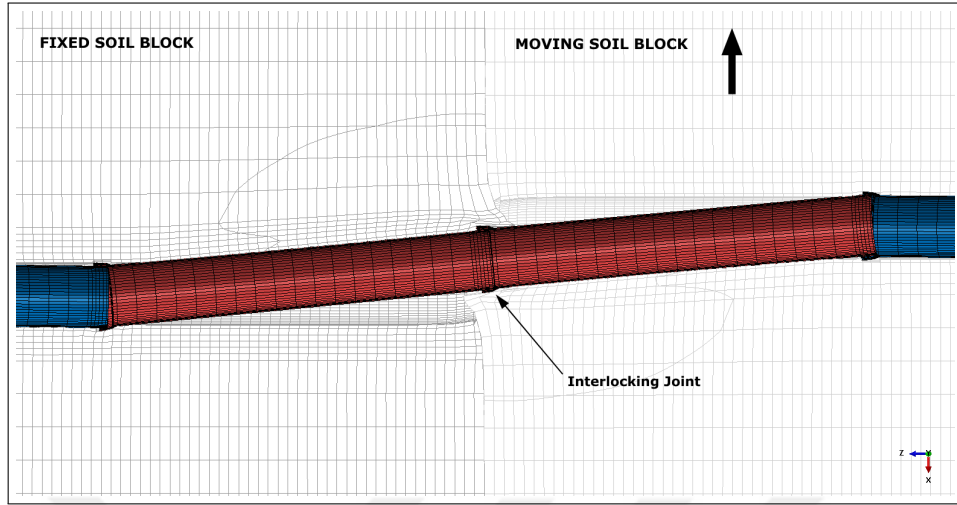


Figure 5.5. Cross-section visualization of the joint interlocking phenomenon from top view. The two adjacent pipe segments rotate and behave as a single unit. Pullout failure was prevented at the affected joint for the shown example case where $\beta = 90^\circ$, $\gamma = 1$, and soft clay deposit was used.

5.2.3. Axial Joint Separation versus Fault Displacement History for $\beta = 90^\circ$ Scenarios

Figure 5.6 shows the normalized ΔU_{rel} values ($\Delta U_{rel}/U_{crit}$, abbreviated as "Norm. Rel. Axial Disp." in figures) on y-axis to provide a direct comparison between the cases for both pullout failure criteria. For the scenarios where $\beta=90^\circ$, the normalized relative axial displacement of the joint elements follow two nearly identical trends governed by their γ value. Moreover, Figure 5.6a shows that the $\gamma = 0.5$ cases lead the pullout failure when subjected to standard failure criteria. For the less conservative failure threshold shown in Figure 5.6b, both of $\gamma = 0.5$ cases again suffered pullout failure but accommodated nearly the full 1 m of fault displacement. Both $\gamma = 1$ cases accommodated 1 m of fault displacement for the two failure thresholds without any failure. Among the four cases presented in Figure 5.6, models with soft clay deposit led the axial joint separation before medium dense sand models for both failure thresholds.

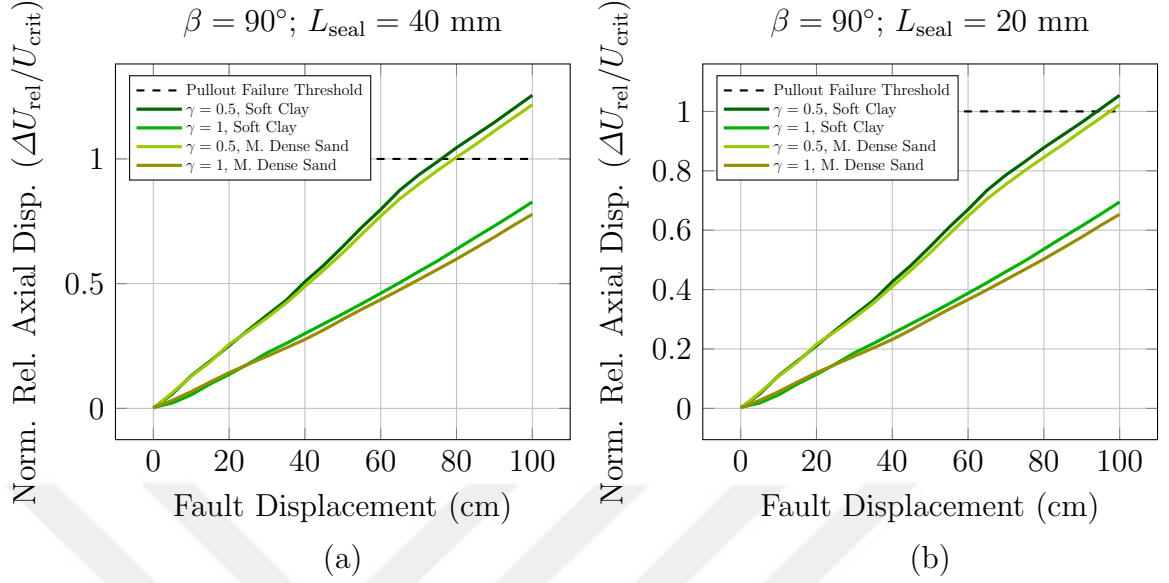


Figure 5.6. Normalized axial relative displacement ($\Delta U_{rel}/U_{crit}$) as a function of fault displacement for $\beta = 90^\circ$ cases. (a) Standard pullout failure criterion. (b) Less conservative pullout failure criterion. The black dashed line represents the threshold for pullout failure ($\Delta U_{rel} = U_{crit}$).

5.2.4. Axial Joint Separation versus Fault Displacement History for $\beta = 67.5^\circ$ Scenarios

Figure 5.7 compares normalized axial joint separation vs fault displacement history for the $\beta = 67.5^\circ$ cases under two different pullout failure criteria. Same as the $\beta = 90^\circ$ cases, the normalized relative axial displacement of the joint elements are again governed by their γ value throughout the 1 m of fault displacement process. Further suggesting that the axial separation of joint is highly influenced by the γ of the scenario for the $\beta = 67.5^\circ$ cases as well. This was observed in cases ($\beta = 67.5^\circ$, $\beta = 90^\circ$) where relatively less longitudinal displacements and more rotations are exerted on the the pipeline system compared to $\beta = 45^\circ$ cases. Moreover, suggesting that this could be interpreted as a result of interlocking segments preventing higher rotations as well as minimizing the axial joint separation. This idea can be supported by the differences in observed joint rotations presented in Figure 5.4, as the $y=0.5$ cases accommodated significantly more rotation compared to $y=1$ cases.

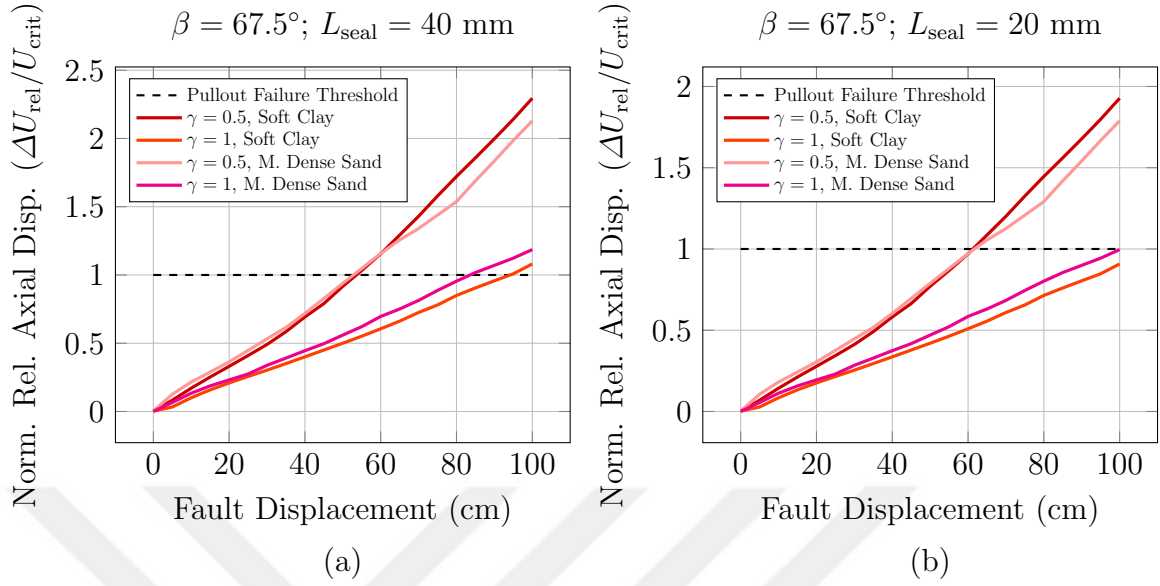


Figure 5.7. Normalized axial relative displacement ($\Delta U_{rel}/U_{crit}$) as a function of fault displacement for $\beta = 67.5^\circ$ cases. (a) Standard pullout failure criterion. (b) Less conservative pullout failure criterion. The black dashed line represents the threshold for pullout failure ($\Delta U_{rel} = U_{crit}$).

5.2.5. Axial Joint Separation versus Fault Displacement History for $\beta = 45^\circ$ Scenarios

The Figure 5.8 shows that $\beta = 45^\circ$ cases have a complex behavior. This is likely caused by the less perpendicular and more longitudinal motion changing the critical joint governing the pullout motion. The effect of pipes segments fully separating and dropping on the below native soil bedding after complete axial pullout is also available in these figures. This effect can be observed from the linear trends after the point of failure in every case indicating the loss of contact and influence of joint geometry.

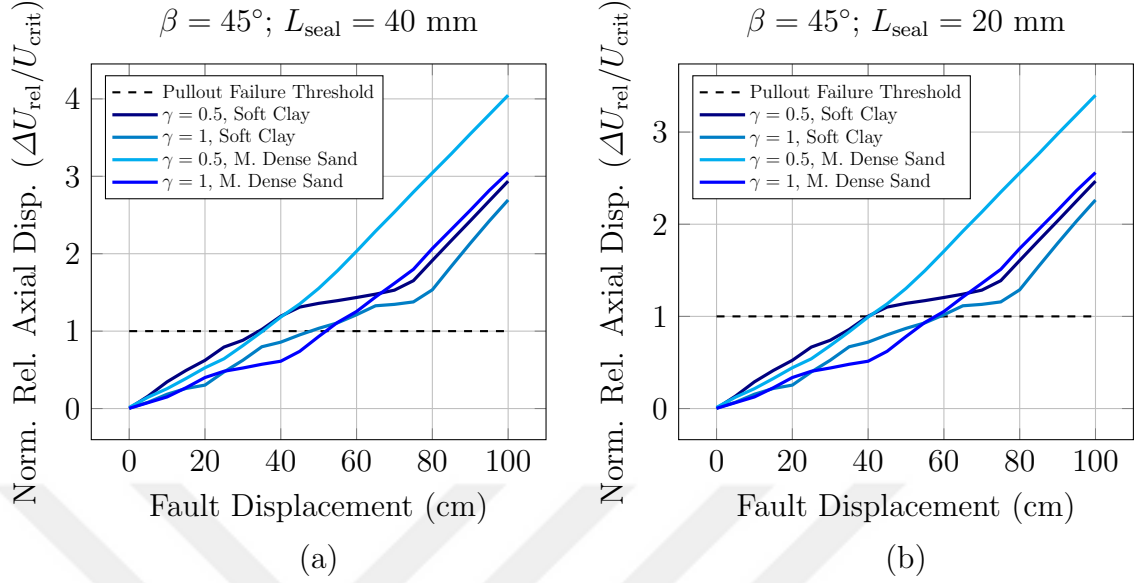


Figure 5.8. Normalized axial relative displacement ($\Delta U_{\text{rel}}/U_{\text{crit}}$) as a function of fault displacement for $\beta = 45^\circ$ cases. (a) Standard pullout failure criterion. (b) Less conservative pullout failure criterion. The black dashed line represents the threshold for pullout failure ($\Delta U_{\text{rel}} = U_{\text{crit}}$).

5.2.6. Pullout Performance of Pipelines As a System

The following Figure 5.9 and Figure 5.10 provides the distribution of relative joint displacement across the pipeline segments after 1 m of fault displacement has been applied for $\gamma = 0.5$ and $\gamma = 1$ cases respectively. The connections are individually named in alphabetical order starting from the fixed soil block.

Overall, the Figure 5.9 reveals that the highest displacement typically occurs in the central portion of the pipeline, more specifically at the two involved joints "C" and "D". For both $\beta = 90^\circ$ cases, the joint that is on the other side of the fault relative to the failed joint is critically close to failure. While in $\beta = 67^\circ$ cases, this second joint has failed for the first threshold in medium dense sand cases and failed for the second threshold as well in soft clay cases. $\beta = 45^\circ$ cases on the other hand show that both of these joint have suffered pullout failure.

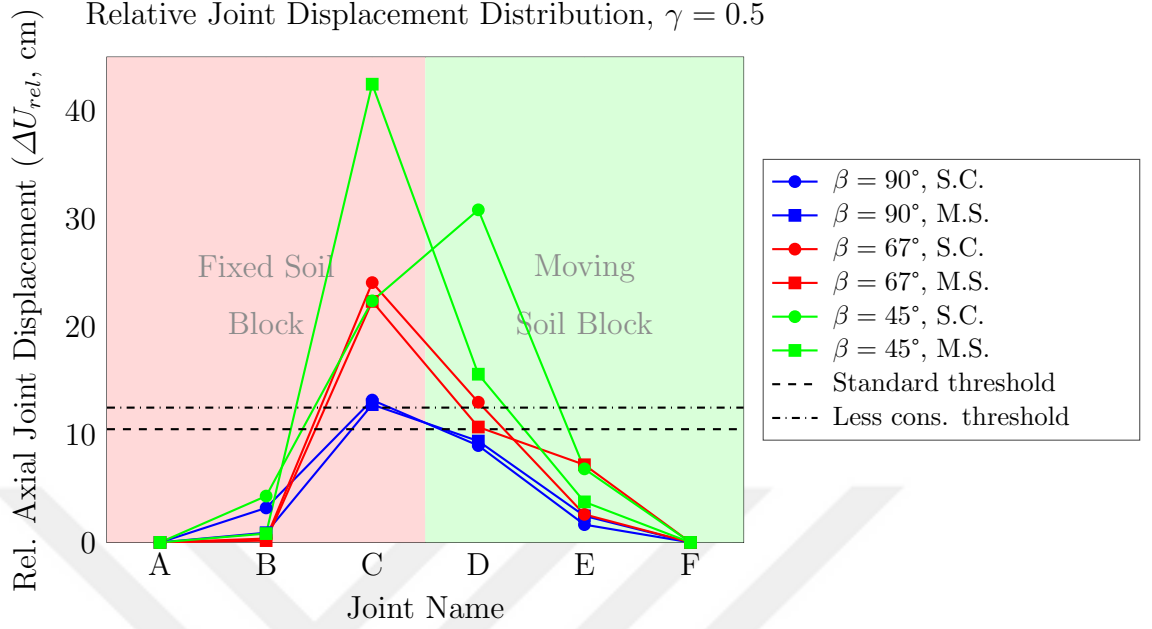


Figure 5.9. Relative joint displacement for different configurations. The red shaded area represents the fixed soil block (joints A-C), and the green shaded area represents the moving soil block (joints D-F).

Figure 5.10 shows a clear visualization of the effect of interlocking segments in $\gamma = 1$ cases. This can be confirmed as the figure shows that the highest displacement does not occur on central joint "D", the where The fault intersection between the fixed block and moving block crosses. Instead, the significant portion of the axial separation joint elements occurs at the nearest two joints "C" and "E". This behavior can be observed in every case presented in this figure. The involved joint "D" avoided pullout failure for both thresholds except for the case where $\beta = 45^\circ$ and soil type is medium dense sand.

The relative axial joint displacement of the elements that belong to interlocking joint D increases as the β value increases. This is attributed to longitudinal forces dominating the motion. This results in less rotation in pipeline, which can be interpreted as among the key factors influencing the phenomena such as the determinant γ .

In cases where $\beta = 90^\circ$ and $\beta = 67.5^\circ$, the joint on the fixed side governs the pullout motion and eventually the failure. While in $\beta = 45^\circ$ cases, the influence of soil starts to appear. In this regard, the effect of different soil models on joint separation can be best observed from this figure as well by looking at the $\beta = 45^\circ$ cases. In medium dense sand model, the joint on fixed side governs the pullout failure, whereas in the soft clay model this is done by the joint on the moving soil block.

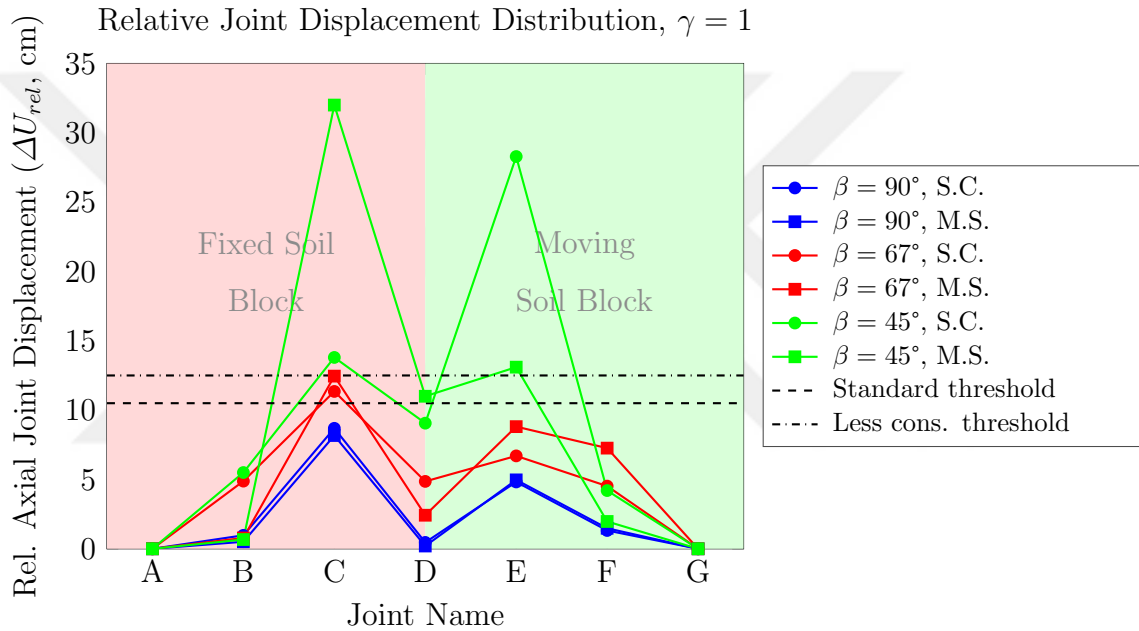


Figure 5.10. Relative joint displacement for different configurations. The red shaded area represents the fixed soil block (joints A-C), and the green shaded area represents the moving soil block (joints E-G).

The total number of pullout failures at the end of the 1 m of fault displacement also showed a trend between different cases. $\beta = 90^\circ$ simulations either completed with no pullout or limited to one, $\beta = 67.5^\circ$ scenarios varied between one or two pullouts per simulation, and $\beta = 45^\circ$ simulations finished with two or three pullouts. This observation suggests an inverse relationship between the number of pullouts and the fault crossing angle. Where narrower β angles exert higher longitudinal motion on the pipeline segments which leads to more pullout failures. The Table 5.2 presents the summarizes the total amount of pullout failures at the end of series of numerical

analyses investigating the pullout failure.

Table 5.2. Fault crossing cases and corresponding number of joint failures at the end of 1 m of fault displacement for the two pullout failure thresholds. Values that correspond to relaxed criterion refers to the less conservative threshold defined earlier in the text.

Case No.	Fault Crossing Angle (β°)	Fault Crossing Position (γ)	Type of Soil Deposit	Number of Joints Failed (Standard Criterion)	Number of Joints Failed (Relaxed Criterion)
1	90	0.5	Soft Clay	1	1
2	90	1	Soft Clay	No Failure	No Failure
3	90	0.5	M. Dense Sand	1	1
4	90	1	M. Dense Sand	No Failure	No Failure
5	67.5	0.5	Soft Clay	2	1
6	67.5	1	Soft Clay	1	No Failure
7	67.5	0.5	M. Dense Sand	1	1
8	67.5	1	M. Dense Sand	1	No Failure
13	45	0.5	Soft Clay	2	2
14	45	1	Soft Clay	2	2
15	45	0.5	M. Dense Sand	2	2
16	45	1	M. Dense Sand	3	2

5.2.7. Best and Worst-Performing Configurations Based on Pullout Response

Figure 5.11 illustrates three cases with different pullout performances after 1 m of displacement is applied. Subfigure 5.11a presents the most favorable configuration among the twenty simulations with $\beta = 90^\circ$, $\gamma = 1$ and medium dense sand. In this

scenario, none of the joints exceeded either the standard or the less conservative pullout thresholds under significant fault movement. Moreover, this could indicate that under such scenarios in real life, the pipeline can potentially stay operational.

The Subfigure 5.11b shows the most premature failure configuration among all tension inducing cases in terms of the maximum accommodated fault displacement prior to pullout failure. The case is characterized by $\beta = 45^\circ$, $\gamma = 0.5$, and soft clay soil deposit with pullout occurring at a relatively small fault displacement of 33 cm. The effect of narrower β angle inducing more longitudinal displacement as well as less rotation can be observed here along with fault crossing position (γ) inducing stresses on the middle of the segment. In this scenario, the effect of $\gamma = 0.5$ on rotation was not governing the pullout performance due to narrower β angle.

The case configuration among pullout cases which induced the most repairs to pipeline in terms of cumulative damage at the end of 1 m of fault displacement is presented in Subfigure 5.11c. Where the $\beta = 45^\circ$, $\gamma = 1$ and soil deposit is of medium dense sand. The effect of γ was again not governing the overall kinematic response of the system because of high longitudinal displacement exerted on the pipeline. In this scenario, the pipeline has experienced three axial pullout failures in total. This indicates that at least two pipe segments would require readjustment or replacement for further operation of the system.

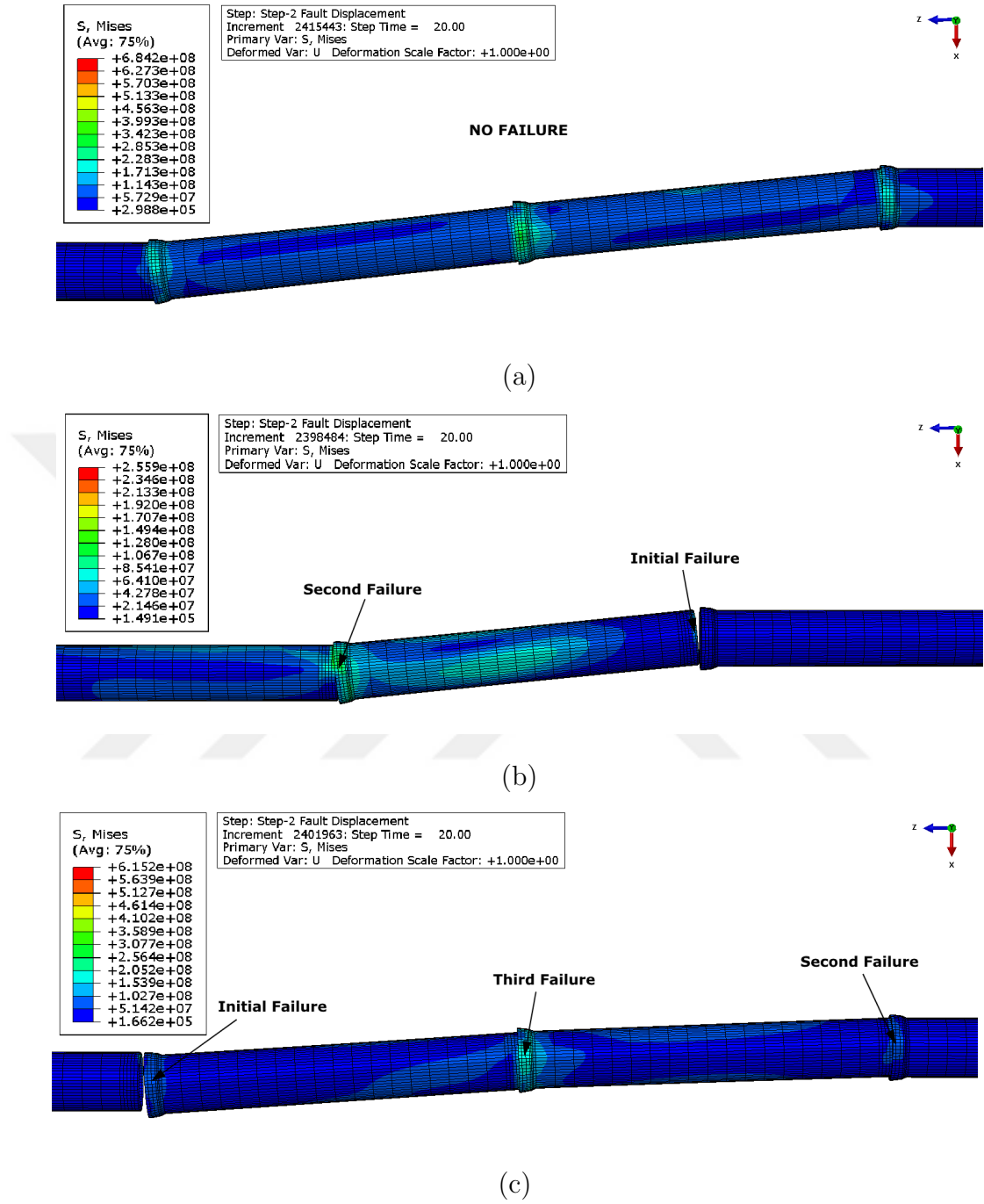


Figure 5.11. von Mises stresses and pipeline performance in three cases after 1 m of fault displacement: (a) The most resilient configuration, showing no pullout failures ($\beta = 90^\circ$, $\gamma = 1$, MS); (b) The earliest failure among tension-inducing cases at 33 cm fault displacement ($\beta = 45^\circ$, $\gamma = 0.5$, SC); (c) The case of most cumulative pullout damage featuring three joint separations ($\beta = 45^\circ$, $\gamma = 1$, MS).

5.3. Compression Inducing Fault Crossing Vector ($-\delta$)

This section provides the results of eight cases where the surface faulting mechanism has negative δ and induces compression on the pipeline system. As mentioned before, the simulations in this part are run until the ultimate stress of the DI material is reached. This is because the thesis does not cover the damage mechanism of DI beyond 460 MPa (crack propagation, necking, rupture characteristics, etc.). Moreover, it was assumed that the simulations lose their accuracy beyond the point of rupture. Another reason is that the models are not provided the damage information for DI, and the joint elements (mainly the end of the spigot) are subjected to extreme stress accumulations as the simulation progresses. As a result, the solver struggles to maintain the stable time increment around the target value, leading to a gradual decrease. Moreover, the simulation time increases significantly. For these reasons, a conservative approach has been adapted in this part of the results as the pipeline system in these cases are assumed to be failed by the crushing of the joint elements at 460 MPa. However, it should be noted that the joints might continue functioning in practice, because the rupture does not first appear at critical areas such as near the sealing zone in the simulations.

In all cases subjected to compression-inducing fault movement ($-\delta$), 460 MPa is consistently reached at the spigot elements of the pipeline. This indicates a clear trend of localized joint crushing failure under compressive joint loading. The behavior reflects the susceptibility of the spigot component to compressive stress concentration, which can be attributed to its relatively thinner geometry and constrained interaction with the bell in these scenarios. As mentioned earlier in Chapter 4, all simulations were initiated with the spigot fully inserted into the bell. Moreover, the joints were at their maximum engagement length at the beginning of the analysis. This initial condition eliminated the possibility of axial separation by the selected β angles. Therefore, no pullout failure was recorded in any of the eight simulations within this group.

5.3.1. Influence of β Angle and γ on Stress Distributions

An observation was made that all joint elements bound together as a result of the induced compression on the pipeline, and stresses start to accumulate in these locations. These stresses transferred between the interface of two joint elements (referred to as binding zones from now on) on each joint. As fault displacement progressed, these binding zones became localized sites for stress accumulation and became links between internal redistribution of axial forces along the whole pipeline. Stress concentrations appear to propagate in a wave-like manner shown in Figure 5.13 between the binding zones, and created bands of high stress separated by lower-stress regions.

However, these stress waves were more prominent in the $\beta = 67.5^\circ$ cases. Due to the normal component of the imposed displacement, the pipeline exhibited more rotational behavior around the joint areas. As a result, stress concentrations were more localized to smaller zones around the spigot's circumference, typically on the side where the spigot made angular contact with the bell.

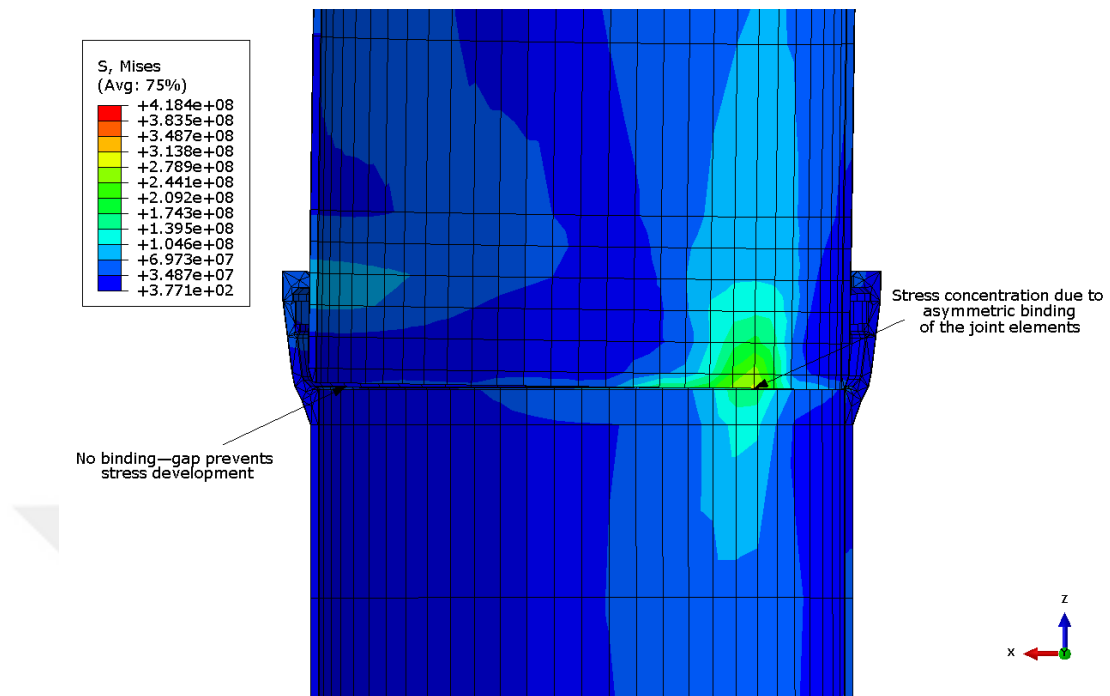


Figure 5.12. Top-down view of the joint cross-section illustrating localized stress accumulation resulting from abrupt binding angle.

In contrast, $\beta = 45^\circ$ cases induced relatively less normal displacement and caused binding zones to have a relatively greater area along the circumference compared to $\beta = 67.5^\circ$ cases. Moreover, stresses accumulated to distribute more evenly around the near-full circumference of the spigot ends.

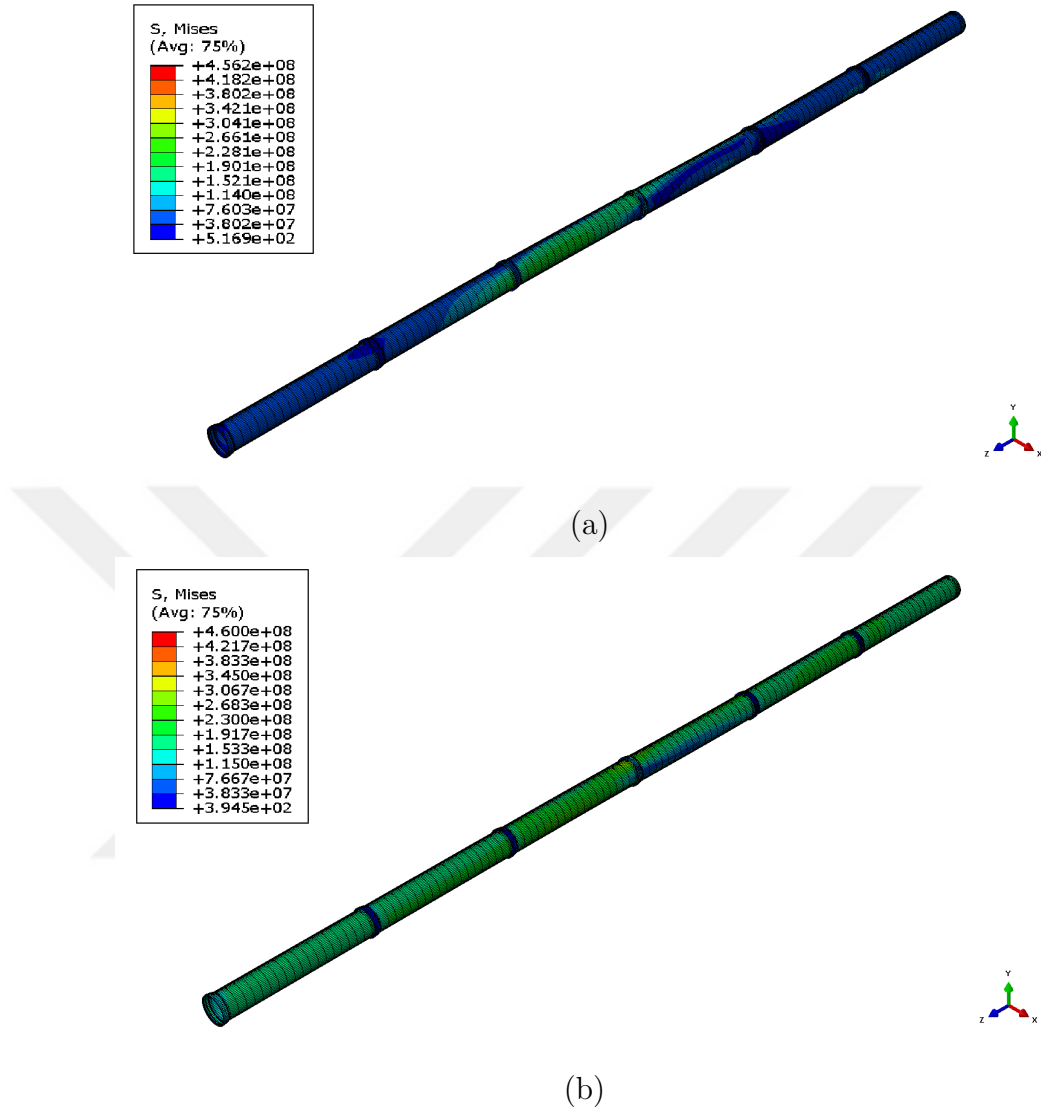


Figure 5.13. von Mises stress distribution along the pipeline at failure for two fault angles: (a) $\beta = 67.5^\circ$, (b) $\beta = 45^\circ$. The wave-like stress accumulation observed in (a) is more localized due to dominance of normal displacement, whereas in (b), the stress is distributed more uniformly around the circumference of the spigot ends due to increased axial displacement demand.

Despite these differences in stress distribution patterns, the area of the binding zones did not govern the maximum fault displacement that could be accommodated before failure between cases with different β values. This is because, while $\beta = 45^\circ$ scenarios showed more uniformly distributed stresses along the ends of the spigots,

they also experienced higher axial demands compared to $\beta = 67.5^\circ$ cases. This can be explained by the vector decomposition of the applied fault displacement, as a 45° vector results in a greater longitudinal displacement component, which imposes more direct axial load on the system.

The difference in accommodated fault displacements between the $\gamma = 0.5$ and $\gamma = 1$ cases at $\beta = 67.5^\circ$ can be attributed to the relative rotation of the joint elements, which results from the normal displacement. In previous observations where pullout was the main concern, the $\gamma = 0.5$ cases exhibited greater rotation earlier in the simulation. A similar behavior is observed here in the compression cases as $\gamma = 0.5$ models exerted more pronounced rotation. As a result, the joint elements bind in a more localized and less uniform manner, leading to early stress concentrations. In contrast, for the $\gamma = 1$ cases, the affected joints located directly at the fault crossing engage more uniformly at the fault displacement level where $\gamma = 0.5$ cases had already failed. This more distributed stress pattern caused by uniform binding could be the reason for delay of the material failure. Similar to other phenomena, this effect was not present in $\beta = 45^\circ$ cases due to insufficiently induced normal displacements on the pipeline.

Table 5.3. Summary of compression-inducing fault movement cases. The table presents the corresponding fault displacement at which failure occurred for each case.

Case No.	Fault Crossing Angle β°	Fault Crossing Position (γ)	Type of Soil Deposit	Fault Displacement at Failure (cm)
9	67.5	0.5	Soft Clay	15
10	67.5	1	Soft Clay	20
11	67.5	0.5	M. Dense Sand	15
12	67.5	1	M. Dense Sand	20
17	45	0.5	Soft Clay	10
18	45	1	Soft Clay	10
19	45	0.5	M. Dense Sand	10
20	45	1	M. Dense Sand	10

It should also be noted that the two soil models had no distinct contributions to pipeline response in compression-inducing scenarios. Across all cases, failure consistently occurred at the spigot ends, and the location and nature of stress accumulation showed similar patterns in both soft clay and medium dense sand. This suggests that, the compressive response of the pipeline system is more strongly governed by geometrical parameters such as fault crossing angle (β) and position (γ), than the differences in stiffness, shear strength or the friction between the two pipe-soil interface of the two surrounding soil models.

6. CONCLUSION AND DISCUSSION

This chapter concludes the study and states the outcomes obtained from the main FEM models. Final remarks are given regarding the limitations encountered during the study as well as the use of explicit solvers in earthquake engineering related problems. Lastly, the contribution of the thesis to literature and possible future expansions are discussed.

The thesis investigated the structural behavior of ductile iron pipelines subjected to lateral strike-slip fault displacement, with a focus on the system's performance under different configurations. The adapted approach included a literature review, model validation, and twenty finite element simulations. Main concern of the study was numerically simulating 1 m of perfectly lateral fault displacement on a TYTON jointed buried segmented pipeline to establish their structural response.

6.1. General

Initial outcomes demonstrated that medium dense sand, representing a stiffer soil condition, induced greater stress and rotation on the pipeline body compared to soft clay. Nonetheless, under all tension-inducing ($+\delta$) scenarios, these stresses remained below the material's yield threshold. The sand model also exhibited relatively higher frictional grip on the pipeline's outer surface as it was defined in modeling phase. While the significant effect of friction was only observable in in $\beta = 45^\circ$ cases likely due to increased axial displacements, no substantial or generalizable impact of friction on the overall pipeline response was identified for other β angles. However, it is hypothesized that friction may play a role in influencing the specific joint at which pullout failure initiates as well as the cumulative number of pullouts in a system in cases where higher axial displacements govern the response. In cases involving yielding of DI, plastic deformation was mostly concentrated at the joints (particularly at the spigots) while the pipe bodies mostly remained in the elastic range. Pullout failures were limited to a

maximum of three joints per case, and in material failure scenarios, plastic deformation often developed at multiple joints prior to rupture. In total, in all 8 scenarios where the soil induced tension on the pipeline ended with pullout failure for the standard failure threshold. On the other hand, 2 of these cases ended with no failure for the less conservative failure threshold. Among the 4 scenarios where the fault crossed the pipeline perpendicularly, 2 cases ended with pullout failure for both the standard and the less conservative failure thresholds. For the 8 $(-\delta)$ scenarios where compression is induced on the pipeline, the system has reached the ultimate strength of the DI in every simulation. Maximum recorded fault displacement prior to failure in these cases was 20 cm.

The fault crossing vector (δ) ultimately governed the type of pipeline failure to be encountered. Moreover, the positive as well as the neutral (perpendicular) δ led to either pullout failure or no failure. It was observed that as the fault crossing angle (β) approaches 90° , local stress concentrations increase, whereas narrower angles lead to axial separations. Whereas $-\delta$ caused joint crushing failure on the pipeline. Under compression, the pipeline undergoes material failure, while tensile forces primarily lead to joint pullout.

6.2. Tension-Inducing Cases

For cases where soil displacements induced tension on the pipeline, the pullout failure under lateral surface faulting localized on a segmented pipeline are strongly dependent on the parameters: (a) fault crossing vector (δ); (b) fault crossing angle (β); (c) critical expansion limit of the joint (U_{crit}); (d) fault crossing position (γ). It was clearly observed that as the relative angle of the fault to the pipeline (β) approached 90° , the TYTON joint was able to accommodate more fault displacement. On the contrary, narrower angles caused more axial displacements and therefore more premature pullout failures. Pullout failure behavior was significantly influenced by the adopted failure criterion. As the fault orientation becomes more perpendicular to the pipeline axis, the role of joint expansion capacity (U_{crit}) becomes more pronounced in governing pullout

resistance. Additionally, the standard (more conservative) criterion resulted in earlier failure compared to the less conservative one. By allowing an additional 2 cm of axial separation, the less conservative pullout threshold resulted in up to 20 cm of additional fault displacement tolerance in the $\beta = 90^\circ$ cases. This highlighted the sensitivity of pipeline performance assessments to the assumed failure definition. The crossing position of fault (γ) on pipeline's body stood out as another important parameter. In cases where the fault interface crossed the middle of the joint perpendicularly, the pipeline was able to accommodate the whole 1 m of fault displacement with no failures. This was due to the behavior exerted by the interlocking segments, which is when the two segments rotate together as a single, elongated unit. The phenomenon effectively prevented relative axial displacement and localized the stress around the bell entry without exceeding the yield limit, and inhibited pullout failure. This finding suggests that joint interlocking is a desirable factor in pipeline resistance to pullout failure during lateral strike slip surface faulting. The elements of the affected joint consistently exhibited this behavior where the fault crossed the pipeline from a joint ($\gamma = 1$) and the displacement vector (δ) was neutral or positive. As for the rotation of the TYTON joints in tension inducing cases, by observing the consistent trends in joint rotation magnitudes at failure, the relative influence of the examined parameters was listed. Among them, fault crossing angle (β) exerts the most significant impact on joint rotation, followed by fault crossing position (γ). Meanwhile, the type of soil model appears to have a lesser but still noticeable effect on rotational behavior in cases of pullout failure. It is worth noting that manufacturer-specified rotational limits for TYTON joints typically fall within the range of 5–6 degrees. However, simulation results revealed that pullout failure can occur well beyond these values. In one case, a joint successfully accommodated up to 8.4° of relative rotation before failure was initiated. This discrepancy may be due to the conservatism of manufacturer guidelines, which likely include safety factors or represent limits for pullout initiation rather than catastrophic failure. Moreover, even higher rotational capacities might be achievable under altered fault geometries, and actual joint performance under seismic loading may exceed standard assumptions under certain conditions.

6.3. Compression-Inducing Cases

In compression-inducing fault scenarios, all cases consistently exhibited localized joint crushing failure at the spigot elements by reaching the ultimate stress limit (460 MPa) of DI. This behavior highlights the spigot's vulnerability under compressive loading due to its thinner geometry and constrained contact with the bell. In these scenarios, no separation (therefore pullout failure) was observed on joints because of their fully engaged initial position combined with the direction of the fault displacement vector. Instead, these conditions promote stress transfer through binding zones at joint interfaces. Stress accumulated in wave-like patterns between these zones, particularly pronounced in $\beta = 67.5^\circ$ cases where greater joint rotation caused more localized stress concentrations. In contrast, narrower angled $\beta = 45^\circ$ cases exhibited more uniform circumferential stress distributions but experienced earlier failure due to increased axial demands. The same relation between β angle and the accommodated fault displacement before failure is also observed in pullout cases, as previously mentioned, the narrower angles again caused more axial displacements which caused earlier failure. Based on these findings, it can be concluded that in lateral strike-slip surface faulting scenarios, buried pipelines with unrestrained TYTON joints show better performance when the fault crosses the pipeline perpendicularly. The fault crossing position also played a critical role in compression inducing cases; $\gamma = 0.5$ cases again rotated more, which lead to non-uniform binding and earlier failure. On the other hand, $\gamma = 1$ configurations showed less rotation and consecutively more evenly distributed stress that delayed failure. This effect was only observable in $\beta = 67.5^\circ$ cases due to higher normal displacements. Based on this information, another similarity can be observed between compression and tension inducing cases, as the system was able to accommodate more fault displacement in cases where the fault crossed the pipeline through the middle of the joint ($\gamma = 1$) compared to when it crossed through the middle of the segments ($\gamma = 0.5$). Notably, the soil type had a minimal impact on the response in these cases, which indicates that pipeline performance is primarily governed by fault geometry parameters (β and γ) among the other studied parameters under compressive loads.

6.4. Performance Extremes: Desired vs. Critical Cases

Overall, among the twenty FEM models, the most favorable configuration was identified as $\beta = 90^\circ$, $\gamma = 1$ with medium dense sand with no joints exceeding the two pullout thresholds. This suggests that the pipeline may remain operational under similar real-life conditions. Among the tension-inducing cases, the most premature failure occurred in the $\beta = 45^\circ$, $\gamma = 0.5$ configuration with soft clay, where pullout was observed at just 33 cm of fault displacement. In terms of cumulative damage in tension cases, the case with $\beta = 45^\circ$, $\gamma = 1$ and medium dense sand resulted in the most extensive damage after three pullout failures after 1 m of displacement. Among the compression-inducing cases, the most favorable performance was observed in configurations with $\beta = 67.5^\circ$ and $\gamma = 1$. The pipeline accommodated the most fault displacement (20 cm) without exceeding failure criteria in these cases. In contrast, all cases with $\beta = 45^\circ$ demonstrated the least favorable response and consistently resulted in early material failure due to increased axial displacements.

6.5. Limitations Encountered During the Study

Throughout the progression of this study, numerous challenges have been encountered due to hardware limitations and consecutive time constraints. Due to the significant computational time required for each simulation, the number of cases included in the parametric study was limited. Because of this constraint, the representative scenarios were carefully selected to ensure meaningful comparisons. The frequency and scope of field output requests had to be restricted to maintain manageable file sizes and avoid excessive memory usage during post-processing. As a result, some variables were recorded at wider intervals which resulted in less accurate results in some results. Another example is that the numerical modeling of hyperelastic gaskets were excluded from the main analyses due to their requirement of extremely high mesh resolution. A similar example is that the bell and spigot elements were unable to be modeled with higher mesh resolution due to mesh dependency of the contact algorithm requiring the 2:1 resolution to propagate through the whole soil domain. In addition to these issues,

the mentioned additional mesh elements need to be multiplied for each pipe joint that is going to be included in the simulations, which drastically increases the required time as well as the effort. Moreover, these issues are all primarily connected to hardware limitations, which forced the author to establish a balance for the whole domain of the model without sacrificing from numerical accuracy.

Another limitation was due to the scope of the thesis not including the post-rupture behavior of ductile iron (DI). As a result, the compression-induced cases had to be terminated shortly after the onset of material failure to maintain accuracy and prevent the solver from slowing down, which led to a more conservative assessment of the pipeline's performance in these cases.

6.6. On the Use of Explicit Solvers in Earthquake Engineering

A comment can be made regarding the common perception that explicit solvers in FEM analyses are typically for cases where inertial effects dominate (high-damage scenarios, impact, creep, explosion simulations etc.). Although, throughout the process in this thesis, it has been observed that the explicit solvers are also capable of effectively simulating overly specific scenarios that are related to earthquake engineering. They can also overcome many convergence issues otherwise encountered in implicit analyses. On the other hand, establishing numerical models for explicit solvers is relatively far more complex, as they require consideration of additional factors such as energies or inertia developing within the system and the stable time increment etc. Explicit solvers also require expert knowledge to avoid numerical issues which makes the user more prone to encounter errors if model settings are not carefully managed. For these reasons, explicit solvers, and more specifically Abaqus, are not commonly preferred in such fields.

Many software tools designed for commercial use in civil and earthquake engineering (e.g., SAP2000, ETABS) use well-optimized implicit solvers, and they can internally handle convergence and stability problems without needing the user to con-

sider various subjects. Such software is easier for the user to learn and work with, but their downside is that they are specifically designed for commonly encountered problems in their own fields only. In this regard, a comment can be made that explicit solvers are robust and comprehensive tools for specific problems such as the buried segmented pipeline example presented in this thesis. Using software that utilizes explicit solvers, highly detailed realistic FEM models can be developed for many earthquake engineering related problems.

6.7. Final Remarks and Contribution for Possible Future Expansions

In possible future expansion of this thesis, hybrid segmented pipeline joints can be simulated to investigate the effects of various locking mechanisms on pipeline performance. Highly nonlinear simulations involving reverse faulting (leading to extreme compression in the segmented pipeline joints) could be simulated with post rupture effects. Site-specific case studies could be done by incorporating real earthquake ground motion recordings to simulations. This would allow for a more realistic assessment of pipeline behavior under actual seismic loading conditions for many types of PGDs. Modified Mohr-Coulomb plasticity definitions can be introduced to include the post-peak softening behavior of certain type of soil. Therefore, even more realistic results can be obtained regarding soil deformation. The influence of various low-density back-fill materials (e.g., pumice, expanded polystyrene geofoam), which are commonly used to reduce loads and friction, can also be investigated by modeling a trench through partitioning methods. This approach could allow for a clearer comparison of the effects of different levels of stiffness and friction on pipeline response. In addition, the size of the future models can be adjusted to simulate entire pipelines, thus effects that require more advanced tools such as TGD or combined effects of multiple PGDs can be considered. For example, this may include the effects of liquefaction and faulting combined, or abrupt local changes in the direction of fault planes along the pipeline etc.

The effect of gaskets and crack propagation can be added in models such as those in the thesis, as these aspects can be already implemented in smaller scale FEM models. Fluid simulations can also be integrated into these models to obtain leakage numerically, as fluids can be already simulated realistically in other software as a physical body. Besides, assessing pipeline performance in terms of leakage is critical, which is ultimately the main purpose of research for seismically resilient pipeline systems.

In these regards, this study aims to contribute to the groundwork for more complex and comprehensive simulations in the future. Moreover, the methodologies and applications presented in this thesis may serve as a resource for researchers and engineers who seek to develop and implement such advanced models.

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