Chapter 7
Fragility Functions of Gas and Oil Networks

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Abstract The present chapter aims to present and review fragility curves for components of gas and oil system networks. These fragility functions need to be applicable to the specific European context and they should be available for a variety of network components such as buried pipelines, storage tanks and processing facilities (i.e. compression and reduction stations). Based on a literature review, it is found that the available fragility functions are mostly empirical and should be applied to the European context, given the current lack of data needed to validate potential analytical methods of vulnerability assessment. For buried pipelines, fragility relations are reviewed with respect to both wave propagation and ground failure. Existing fragility curves for storage tanks and processing facilities are also critically appraised, according to the modelling assumptions and the derivation techniques (e.g. fault-tree analysis, numerical simulation or empirical relation).

7.1 Introduction

Like other utility systems, gas and oil networks are prone to sustain major physical damages, as proven by past earthquakes. However, besides the lifeline disruption, other consequences often include the pollution of waterways or the onset of fires and explosions. Therefore the accurate vulnerability assessment of gas and oil network components is of critical importance and it is to be focused on the elements

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that are vital to the network operation, namely the piping system, and the storage and processing facilities. In the SYNER-G project (SYNER-G 2009–2013), the identification of specific typologies that may be representative of the European context has been especially carried out through three gas networks (Thessaloniki in Greece, Vienna in Austria and the L'Aquila area in Central Italy).

In the light of post-earthquake damage observations, it is possible to identify the damaging mechanisms for each of the components, depending on the type of seismic action (i.e. transient ground motion or permanent ground deformation) or the component typology (e.g. ductile or brittle pipelines). Relevant intensity measures (IM) and damage scales have also to be selected in order to ensure a proper integration of the fragility functions with the SYNER-G general methodology. However, the physical damage states that can be sampled from fragility curves may also present some shortcomings, since it appears that there is not straightforward correlation between the damage level (i.e. usually based on monetary consideration, like the cost to replace or repair the component) and its immediate consequences on the network operations.

Based on these considerations, a critical review of existing fragility functions for pipelines, storage tanks and processing facilities is then made. Some recommendations are given on which functions can be applied to European typologies, according to a series of criteria: intensity measure, derivation technique (e.g. empirical, numerical, Bayesian, fault-tree analysis), quality of the data used and modelling assumptions. Finally, some limitations and gaps with respect to the identified typologies are discussed.

7.2 Identification of the Main Typologies

The various elements composing the gas and oil transportation and distribution networks can be roughly classified into three categories, i.e. the actual edges of the network (pipelines), the storage tanks and finally the different facilities that perform specific operations such as pressure control or pumping.

7.2.1 Pipelines

The first typological distinction that can be made for pipelines is whether they are buried or elevated above ground, usually on a steel or concrete support. Since buried pipelines are the most typical means of transportation for hydrocarbon products – especially around inhabited areas –, the present chapter will mostly emphasize on this typology.

Table 7.1 Most common types of materials and connections used in the design of buried pipelines

Material type	Connection type
Asbestos-cement (AC)	Arc welded
Cast iron (CI)	Bell and spigot
Ductile iron (DI)	Cemented
Concrete (C)	Riveted
Polyvinyl chloride (PVC)	Rubber gasket
Welded steel (WS)	Gas welded
Medium density polyethylene (MDPE)	
High density polyethylene (HDPE)	

Natural gas networks are operating at various pressures, depending on their scale:

- supra-regional transmission pipelines: these pipelines operate at very high pressure (~100 bar) and present large diameters (up to 1.40 m). Such pipelines can cover large areas (e.g. from West Siberia to Europe, from Norway to France);
- regional transmission/distribution pipelines: these pipes still operate at high pressure (from 1 to 70 bar) and are used to connect local distribution systems;
- local distribution pipelines: these smaller pipelines usually operate in the medium (0.1–4 bar) or low-pressure (<0.1 bar) range.

Therefore the design pressure of the different pipeline types will influence a set of typological features and mechanical standards, namely:

- material type,
- material strength,
- diameter,
- wall thickness,
- smoothness of coating,
- type of connection,
- design flow.

Among the criteria listed above, the material and the connection types are of crucial importance, since they govern the behaviour and the potential failure modes of buried pipelines in the case of an earthquake. Reports from American Lifeline Alliance (ALA 2001) and HAZUS (NIBS 2004) have detailed some of the most common types of materials and connections used for buried pipelines (see Table 7.1). Finally, another relevant criterion to classify pipelines might be the age or the corrosion state, as shown by the poor performance of ancient pipelines in past earthquake events (ALA 2001).

While Table 7.1 details a series of materials that are mostly suitable for water or waste-water transport, pipelines specifically designed for oil and gas are more likely to be made of ductile materials such as steel or PVC. Also, another specific material type is polyethylene (medium or high density, i.e. MDPE or HDPE), which is used in more recent networks due to its high ductility.

For instance, in the case-study areas considered in the SYNER-G project, some specific typologies have been identified, as shown in Table 7.2.

Table 7.2 Main features of some pipeline networks identified in the SYNER-G project

Area	Network	Pressure (bar)	Material	Diameter (mm)
Greece (Thessaloniki)	Transmission	19	WS	100–250
	Distribution	4	PVC	125–160
Austria (Vienna)	Supra-regional	84	WS	200–1,400
	Transmission	16	PVC	–
	Distribution	1	PVC	–
Italy (L'Aquila)	Transmission	64	WS	104
	Distribution	2.5–3	WS/HDPE	25–300/32–400
	Distribution (local)	0.025–0.035		

7.2.2 Storage Facilities

A first distinction can be made between underground and surface storage facilities. Sub-surface facilities for natural gas storage are usually used to balance seasonal variations in demand (i.e. between the heating and non-heating periods). These facilities are located 100 m below the surface and they are usually natural geological reservoirs, such as depleted oil or gas fields or salt caverns.

Aside from underground storage facilities, natural gas is usually stored while in its liquefied state (LNG) in specific LNG tanks: they can represent huge facilities, making them too specific objects for a statistical fragility analysis.

On the other hand, oil storage tanks are atmospheric reservoirs (i.e. vertical cylindrical tanks), which are often categorized by the following features:

- material: steel or reinforced concrete,
- construction type: at grade or elevated,
- anchored or unanchored,
- roof type,
- capacity,
- shape factor: height-on-diameter ratio,
- amount of content in the tank: full, half-full, empty.

The most common typologies are usually based on the material type, the construction type and the anchorage of components. Finally, it should be noted that tanks are just a part of the storages facilities, which include also components like inlet/outlet pipelines or mechanical equipment.

7.2.3 Processing Plants/Stations

In the case of the gas network, processing stations can be first classified according to their function within the system, i.e. compression, metering or pressure reduction.

Compressor stations are used to supply the gas with a given amount of pressure or energy to keep it flowing. They are located along the transmission lines to ensure the transport over long distances and around the storage facilities in order to inject the gas into the distribution network.

Fig. 7.1 Pictures of the outside (*left*) and inside (*right*) of a RE.MI cabin (Esposito et al. 2013)

This type of station usually includes one or more compressor units, auxiliary equipment for secondary functions (i.e. power generation or cooling of discharge gas) and a SCADA (Supervisory Control And Data Acquisition) system. Most of these stations are housed in low-rise buildings and the following features are often used to identify the typologies:

- with anchored or unanchored components,
- within low-rise buildings, made of masonry or reinforced concrete.

For instance, in some European countries like Greece, compressor stations are usually housed in low-rise RC buildings with anchored components.

Metering/Pressure reduction stations (M/R stations) are used to control the amount and quality of the gas flowing through the lines. They usually include a pressure reduction facility in order to set the gas pressure at the required level for industrial or commercial use. Usually such stations include the following features:

- gas pre-heating,
- gas-pressure reduction and regulation,
- gas odorizing,
- gas-pressure measure,
- control through a SCADA system.

These stations have very strong specificities depending on the area where they are located. For instance, in central Italy (i.e. the L'Aquila area), these M/R stations are referred to as RE.MI cabins (i.e. "REgolazione e MIsura" in Italian, see Fig. 7.1) and they are housed in one-storey RC buildings with steel roofs, without any SCADA system (Esposito 2011; Esposito et al. 2013).

These large disparities and the specificities of the operations performed within M/R stations prevent them from being included in the same typology as the compressor stations.

Reduction groups are very local stations that reduce the gas pressure to the level of the distribution network for individual houses. They are the last step of the transmission-distribution chain. They consist in small equipment that can be buried, sheltered in a kiosk or housed within a building. Again, in central Italy, these reduction groups are referred to as GR stations (i.e. "Gruppi di Riduzione" in Italian)

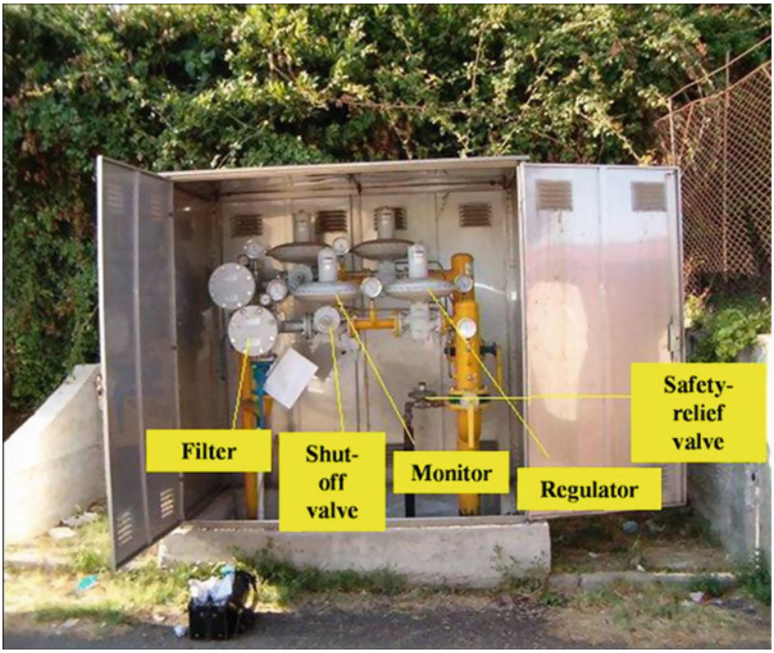

Fig. 7.2 Picture of a GRF station (Esposito et al. 2013)

(Esposito 2011; Esposito et al. 2013). Depending on the amount of gas and level of service pressure required by the end user, and depending on whether or not a final node is included in the system, three types of RG exists in L'Aquila gas network:

- (1) Reduction and measure groups (i.e. GRM), located along the medium-pressure network and directly connected to large-pressure users;
- (b) Reduction groups that are smaller than GRM (i.e. GRU), for medium-pressure users connected to the final node of a medium-pressure system;
- (c) Final reduction groups (i.e. GRF), connected to the low-pressure network (see example in Fig. 7.2).

Pumping stations along oil pipelines can be assimilated to the same typology as gas compression stations, pumps and compressors possessing very similar characteristics and functionalities.

7.3 Description of Damage Mechanisms and Failures Modes

The various elements composing gas and oil networks are sensitive to very different seismic intensity measures (e.g. acceleration or displacement), depending on their very nature (e.g. buried or at-grade elements, ductile or fragile materials…). Therefore the following section is devoted to the description of the various damage mechanisms that can impact the network components.

Table 7.3 Overview of the two main types of damage mechanisms affecting buried pipelines

	Ground failure	Transient ground deformation
Hazard	Surface faulting, liquefaction, landslides	R-waves, S-waves
Intensity measure	PGD (permanent ground deformation)	PGA, PGV, strain
Spatial impact	Local and very site-specific	Large and distributed

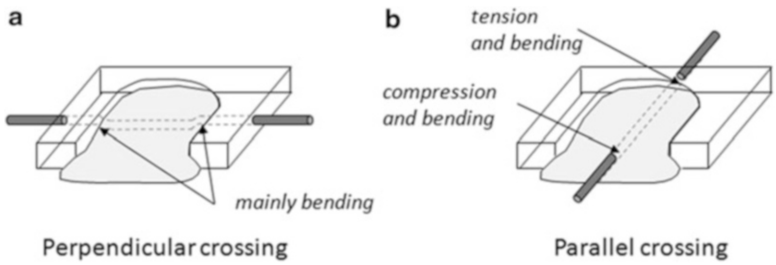

Fig. 7.3 Pipeline damage in (**a**) perpendicular and (**b**) parallel crossings of a lateral spread (Adapted from Rauch 1997)

7.3.1 Damage Mechanisms of Buried Pipelines

Like many other underground components, buried pipelines are very sensitive to permanent ground deformation (resulting from various types of ground failures), in addition to transient ground deformation due to seismic wave propagation: the characteristics of these physical phenomena are summed up in Table 7.3. Indeed, according to Eguchi (1987), past earthquakes have caused significant damages to underground pipelines throughout the world, mainly due to faulting, landslides or liquefaction (Hall 1987).

7.3.1.1 Damage from Permanent Ground Deformation

The first sign of damage to buried pipelines is the 1906 San Francisco earthquake, which resulted in significant fires through the city, due to the rupture of water lines needed by fire-hydrants. Regarding the causes of damage, according to O'Rourke and Liu (1999), only around 5 % of the area that was affected by strong ground shaking was subjected to lateral spreading, yet approximately 50 % of all pipeline failures occurred within one city of these zones, thus showing the high impact of ground failure on pipeline damage.

Damage to buried pipelines induced by permanent ground deformation is usually the main source of failure, as shown by numerous examples of past earthquakes: 1952 Kern County, 1964 Niigata, 1964 Alaska, 2007 Niigata and 2011 Christchurch earthquakes. During the 1971 San Fernando earthquake, the steel pipeline system withstood significant ground shaking, yet it was damaged by abrupt vertical or lateral dislocations or ground ruptures: lateral spreading (Fig. 7.3) induced severe damages during that earthquake (EERI 1986; O'Rourke and Trautmann 1981; O'Rourke 1988). Regarding liquefaction, a good example is the 1964 Niigata earthquake, where the average failure ratio for one of the pipeline systems was as high as 0.97 per km, with all kinds of failure types (e.g. pipe body breaks, weld breaks, joint separations).

7.3.1.2 Damage from Transient Ground Motion

O'Rourke and Ayala (1990) report that a few earthquakes have induced damages to pipelines only by the effect of seismic wave propagation, such as the 1985 Michoacan earthquake, which damaged a large corrosion-free modern continuous steel pipeline, or the 1989 Loma Prieta earthquake. Yet, in most cases, it appears that seismic wave propagation damaged mainly pipelines that were previously weakened either by corrosion or welds of poor quality (EERI 1986). Other events, like the 1994 Northridge, 1995 Kobe, 1999 Kocaeli or 1999 Chi-Chi earthquakes, confirmed the relative vulnerability of piping systems to strong ground motions and the somewhat good performance of recent welded-steel pipes with respect to seismic wave propagation.

As a result, the emphasis is put on the ductility of pipes and the quality of weld when building earthquake resistant piping systems: still, pipe welds or joints seem to be the most vulnerable parts of this component.

7.3.1.3 Identification of Failure Modes for Buried Pipelines

Continuous pipelines like welded-steel pipes usually fail due to compressive strains that induce buckling of the pipe body, or warping and wrinkling of the pipe wall (ALA 2001). This deformation may not generate leakage, yet the modification of the pipe cross-section may produce disruption of the gas/oil flow. A crucial factor for the resistance of continuous pipelines is the quality of the welds, as past studies have shown that pipes constructed before the 1930s with poor quality welds experienced damages mostly at the joint locations.

Segmented or jointed pipelines usually consist of rigid pipe segments (e.g. cast-iron or concrete, which are not used in gas networks) connected through loose or flexible joints. Three main failure modes have been identified for this typology (ALA 2001): tensile and bending deformations of the pipe barrel, excessive rotation of a joint, and pull-out of the joint (Singhal 1984). This pipeline type is however much less frequent in oil/gas piping networks.

Aside from these usual failure modes, a piping system is more vulnerable at discontinuities like pipe elbows, tees, in-lines valves or connections to adjacent structures (storage tanks, racks, facilities, etc.): high stresses are especially concentrating at these anchor points and rigid locations (ALA 2001). Also, corrosion has the effect of decreasing the wall thickness and creating heterogeneous zones that may lead to stress concentrations.

7.3.2 Damage Mechanisms of Storage Tanks

Damage to atmospheric storage tanks (i.e. vertical cylinders) has also been quite extensively documented in past earthquakes (EERI 1986). Damage reports from past earthquakes indicate that unanchored tanks seemed to be the most vulnerable ones, together with vertical cylinders tanks with a large height-to-diameter ratio (EERI 1990). The inherent vulnerability of most storage tanks is also aggravated by the amount of liquid stored, as full tanks are subject to larger lateral forces and overturning moments due to liquid sloshing, which can also damage the tank roof. As a result, failure modes of storage tanks are usually characterized using the following classification (NZNSEE 1986; Kennedy and Kassawara 1989; ALA 2001):

- *shell buckling*: it is one of the most common forms of damage in steel tanks. It is expressed via an outward buckling of the bottom shell courses ("elephant foot") that can sometimes occur over the full circumference of the tank. This phenomenon may lead to the loss of the content due to rupture of the welds, and less frequently to the total collapse of the tank.
- *roof damage*: ground shaking may induce oil sloshing inside the tank. When tanks are full or nearly full, this sloshing motion generates an upward pressure distribution against the tank roof. This may cause a rupture of the joints between the wail and the roof, leading to a spillage of tank contents over the tank walls. Observations from past earthquakes show that floating roofs have generally endured more severe damage than fixed steel roofs.
- *anchorage failure*: many tanks are anchored with steel braces or bolts, but it is still possible that these anchors may be pulled out or stretched by the seismic load. However, the failure of anchoring components does not necessary imply the loss of the tank contents.
- *tank support system failure*: this failure mode is specific for above-grade tanks, elevated by steel columns or frames. Even if the failure of the supporting system often leads to complete loss of contents, this issue is of less concern to large oil storage tanks, which are usually built at grade.
- *foundation failure*: this phenomenon can be common in the case of poor foundation conditions prone to liquefaction, resulting in base rotation and important settlements. In the case of unanchored tanks, tensile stress can also generate uplift displacement of the tank base, separating it from the baseplate.
- *hydrodynamic pressure failure*: ground shaking generates pressures between the fluid and the tank walls, thus resulting in tensile hoop stresses. The induced loads may then lead to splitting of the wall and leakage, especially in the case of steel tanks with riveted joints.
- *connecting pipe failure*: this is one of the most common failure modes that can induce a total loss of the tank contents. The fracture of the pipes at the connections to the tank results from differential displacement between the piping and the tank (uplift displacements, foundation failure).
- *manhole failure*: because of significant stresses against the manhole cover, the latter can fail which results in loss of content through the opening (Fig. 7.4).

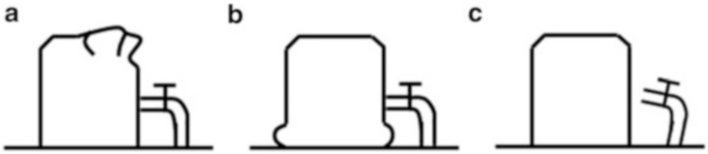

Fig. 7.4 Schematic view of some of the most common failure modes of storage tanks: (**a**) roof damage due to sloshing, (**b**) elephant's foot buckling and (**c**) disconnection of inlet/outlet piping

7.3.3 Damage Mechanisms of Processing Plants/Stations

Reports from previous earthquakes mention limited examples of damaged support facilities (EERI 1986), since it is usually found that modern facilities (compression stations, pumping stations, control stations, etc.) that are built according to seismic codes with anchored equipment exhibit good resistance to ground shaking. The anchorage of subcomponents is especially a crucial point, as unanchored equipment can lead to the rupture of electrical connections or the tipping and sliding of mechanical parts. As a result, support facilities are less documented with respect to their damages and the associated failures modes. However, failure of the various components of these facilities may be used to identify the global damage mechanism:

- *building*: the collapse of the structure sheltering the facility may damage the equipment with falling debris;
- *pump/compressor*: this key element is connected to the piping system and its failure, due to sliding or rocking if unanchored, can generate leakage or breakage of the pipe;
- *electrical/mechanical components*: these miscellaneous components, which are essential for the compressor to operate, can also be damaged if not anchored;
- *electric power supply*: external power can be shut down because of the electric power network disruption, or the connection failure between the power lines and the facility building. However, most facilities are equipped with backup power generators.

Regarding in-line valves, many types are found along the piping network (gate valves, butterfly valves, check valves, ball valves, etc.) and they can be either buried with the pipeline or located in underground concrete vaults. Finally, SCADA equipment includes many components (instrumentation, power supply, communication components, vaults, etc.). For hardware located in metal cabinets, the main observed damages comprise batteries falling over, circuit boards dislodging and gross movement of the cabinet enclosure (ALA 2001). Regarding pressure/flow measuring instruments, ground shaking is likely to induce air bubbles that can provoke false reading.

Table 7.4 Solutions for longitudinal ground strain as a function of incidence angle θ, particle velocity v_p and apparent wave propagation velocity c (St John and Zahrah 1987; Hashash et al. 2001)

Wave type	Longitudinal strain	Maximum longitudinal strain
P wave	$\varepsilon_P = \frac{v_{pP}}{c_P}.\cos^2\theta$	$\varepsilon_P = \frac{v_{pP}}{c_P}$ for $\theta = 0°$
S wave	$\varepsilon_S = \frac{v_{pS}}{c_S}.\sin\theta.\cos\theta$	$\varepsilon_S = \frac{v_{pS}}{2c_S}$ for $\theta = 45°$
R wave	$\varepsilon_R = \frac{v_{pR}}{c_R}.\cos^2\theta$	$\varepsilon_R = \frac{v_{pR}}{c_R}$ for $\theta = 0°$

7.3.4 Key Modelling Issues

This section details some of the specific characteristics that must be accounted for when modelling the different network components in the frame of a numerical analysis.

7.3.4.1 Buried Pipelines

Analytical modelling of the behaviour of buried pipelines submitted to ground motion requires taking the whole [pipeline-soil] system into account. Since inertia forces are not relevant in the case of buried components, the seismic action has to be represented in terms of ground strain: the longitudinal strain is usually acknowledged to have the most impact on the failure of pipelines. While assuming a constant shape for a single surface wave, the peak horizontal ground strain ε_p can be expressed as the following (Newmark 1967; Newmark and Rosenblueth 1971), where v_p is the peak horizontal particle velocity and c the apparent wave propagation velocity with respect to the surface:

$$\varepsilon = \frac{v_p}{c} \tag{7.1}$$

Using this formulation, St John and Zahrah (1987) proposed solution to estimate longitudinal ground strain with respect to incidence angle θ, for different types of waves (see Table 7.4). It can be observed that the longitudinal strain is maximal when the wave incidence is parallel for P- and R- waves, and oblique (i.e. $\theta = 45°$) for S-waves.

The particle velocity can then be assimilated to the PGV, thus providing a relation between the seismic ground motion and the resulting ground strain, allowing for instance back-analysis of past earthquakes to get an estimate of the maximum ground strain. Moreover, it is also possible to obtain a relation between the permanent ground deformation (PGD) and the ground strain, using for instance pre- and post-earthquake photogrammetric analyses: therefore this enables to use a single measure to characterise both phenomena (i.e. transient deformation and ground failure) and to propose a consistent fragility relationship for all types of event (O'Rourke and Deyoe 2004).

Finally, the analysis of the connections between the pipe segments and the influence of the wave incidence angle with respect to the pipe alignment should also be given special care in the fragility analysis.

7.3.4.2 Storage Tanks

In the case of vertical cylinder tanks, one key issue consists in the fluid–structure interaction that may influence the global behaviour of the tank under dynamic excitation (i.e. sloshing of fluid may modify the response of the tank walls and even induce damage). For instance, empirical observations and structural analyses have shown that tanks whose filling level is greater than 50 % are more vulnerable to earthquakes (Salzano et al. 2003), while the height-over-radius ration constitutes also an important factor for the seismic response of tanks.

The presence or not of anchorage of the tank to its base will also greatly condition the outcome of the analysis, since an unanchored tank may be subject to sliding and rotating on its base, making it unusable while the tank structure itself may still be intact. The same comment can also be made regarding the equipment servicing the storage tank (i.e. inlet/outlet pipes): these subcomponents should be considered in the analysis, since they could easily get torn apart from the main structure, rendering it once again unusable.

7.3.4.3 Processing Plants/Stations

Support facilities are usually sheltered in a small building and a first level of analysis consists of only considering the fragility of the building (i.e. if there is extensive damage to the building, the facility is considered non-functional). However, it is the damage to the mechanical or electrical components within the building that should be considered, as they have a fair chance to rupture or get disconnected, especially if they are unanchored. A distinction between the acceleration- or displacement-sensitive components should first be made and the various building response parameters should be estimated at each story (e.g. both floor and roof accelerations or displacements in the case of drift-sensitive equipment). The different configurations of equipment that may exist within a given station would make this type of analysis very case-specific however.

7.4 Review of Existing Fragility Functions and Gaps

This section is devoted to a description of the most common fragility functions that are available in the literature, along with their associated intensity measures and damage scales.

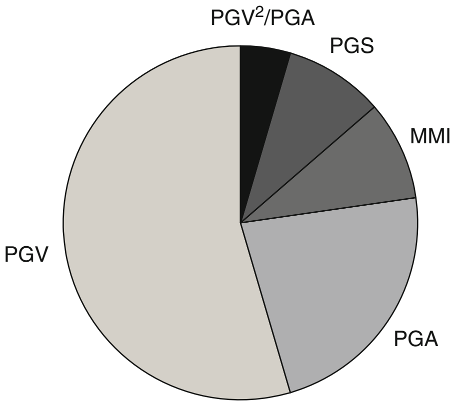

Fig. 7.5 Relative proportions of the IM types used in common fragility functions

7.4.1 Definition of Adequate Intensity Measures

A short review of some commonly used intensity measures is proposed for the different components of oil and gas networks.

7.4.1.1 Buried Pipelines

Existing fragility relations considering the effect of wave propagation consider a variety of intensity measures such as PGA, PGV, PGV^2/PGA, PGS (peak ground strain) or MMI (macroseismic intensity). The proportions of the IMs used in the reviewed fragility functions are summed up in Fig. 7.5.

It is observed that a wide majority of the fragility relations use the PGV as an intensity measure: this choice is in line with the conclusions of Sect. 7.3.4.1, in the sense that there is a direct link between the longitudinal ground strain and the PGV. Recent studies have even started to propose the peak ground strain (PGS) as a more adequate intensity measure. However, the use of PGV still seems to remain popular, due to its straightforward computation, using for instance a recorded signal or a ground-motion prediction equation, as opposed to the ground strain.

It can be also noticed that most fragility relations do not consider the direction of the pipe whereas it is acknowledged that the longitudinal strain is responsible for the failures. This is justified because once used on the distribution network of a study case, these relations are applied to a large number of pipelines that can be assumed to be randomly oriented.

In the case of damage due to ground failure, all existing fragility relations use the permanent ground deformation (PGD) as the intensity measure. However, it is worth noticing that the study by O'Rourke and Deyoe (2004) has established a good correlation between ground strain and repair rate for both transient and permanent deformations, thus also allowing the use of ground strain as an intensity measure in the case of permanent ground failure.

7.4.1.2 Storage Tanks

Past studies on the vulnerability of storage tanks usually propose PGA as the earthquake descriptor used to define the fragility curves. This seems to be a reasonable choice as this acceleration-driven parameter is appropriate to account for the inertia forces inherent to these large and usually tall structures and the liquid contents within the tanks.

7.4.1.3 Processing Plants/Stations

As most facilities are sheltered in a building, a commonly-used parameter is the PGA, since it is widely used to describe the fragility of RC or masonry buildings. Also, the behavior of anchored or unanchored components within the facility seems acceleration-driven and their fragility are indeed expressed with respect to PGA in HAZUS (NIBS 2004).

7.4.2 Definition of Damage Scales

Damage scales are defined for each of the components considered here, in order to identify the possible damage states that have to be included in the analysis.

7.4.2.1 Pipelines

Damage to pipelines is commonly expressed in repair rate (*RR*), i.e. number of repairs per unit of length (usually in km). For a give pipeline tract of length *L* with a given *RR*, the probability to get a total number of *n* repairs over the pipeline length is then estimated through a Poisson distribution:

$$P(N = n) = \frac{(RR.L)^n}{n!}.e^{-RR.L} \tag{7.2}$$

The repair rate does not make any distinction on the type of repair or their severity: this is due to the fact that most of the fragility relations for pipelines rely on empirical data that do not usually include the nature of the repairs. However, according to HAZUS (NIBS 2004), the type of repair or damage depends on the type of hazard: a damaged pipe due to ground failure is more likely to present a break (it is assumed 80 % breaks and 20 % leaks), whereas ground shaking may induce more leak-related damages (e.g. 20 % breaks and 80 % leaks). Finally, using the HAZUS assumption and considering the type of hazard, it is possible to assess the probability to have a pipe break or a pipe leak along the length of the segment (see Table 7.5).

Table 7.5 Proposed damage states for pipeline components

Damage state		Damage description
DS0	No damage	No leak or break
DS1	Leakage	At least one leak along the pipe length
DS2	Failure	At least one break along the pipe length

Table 7.6 Damage states proposed by HAZUS (NIBS 2004) and O'Rourke and So (2000)

Damage state		Damage description
DS1	None	No damage to tank or inlet/outlet pipes
DS2	Slight/minor	Damage to roof other than buckling, minor loss of contents, minor damage to piping, but no elephant's foot buckling
DS3	Moderate	Elephant's foot buckling with minor loss of content, buckling in the upper course
DS4	Extensive	Elephant's foot buckling with major loss of content, severe damage, broken inlet/outlet pipes
DS5	Complete/collapse	Total failure, tank collapse

Table 7.7 Comparison of failure modes and resulting loss of contents (ALA 2001)

Most common damage modes	Repair cost (% of tank value)	Content loss (%)
Rupture of drain pipe	1–2	50–100
Rupture of overflow pipe	1–2	0–2
Rupture of inlet/outlet pipe	1–5	100
Rupture of bottom plate from bottom course	2–20	100
Roof system partial damage	2–20	0–10
Roof system collapse	5–30	0–20
Upper shell buckling	10–40	0–20
Elephant's foot buckling with no leak	30–80	0
Elephant's foot buckling with leak	40–100	100

7.4.2.2 Storage Tanks

Fragility curves from the literature, whether they are empirical or analytical, usually propose the same number of damage states (i.e. five, including "no damage") and very similar definitions (O'Rourke and So 2000; ALA 2001; NIBS 2004; Berahman and Behnamfar 2007). The detailed damage states used by HAZUS (NIBS 2004) and O'Rourke and So (2000) are presented in Table 7.6.

The damage states definition by ALA (2001) is very similar, apart from the inclusion of upper course buckling in DS3 and of inlet/outlet pipes damage in DS4.

However, the damage states presented above are based on direct economic losses (i.e. percentage of the tank replacement cost), whereas Table 7.7 shows that there is no obvious correlation between this criterion and the functionality of the tank.

Thus, a rupture of an inlet/outlet pipe would only generate repair costs of 1–5 % of the total tank value, but this would put the tank completely out of service. Therefore we can assume that, as soon as damage state DS2 (e.g. damage to piping) is reached, the functionality of the tank may be totally lost, at least for a short amount of time.

Table 7.8 Damage states definitions for tank farms, according to HAZUS methodology (NIBS 2004)

Damage state		Damage description
DS1	None	Fully functional
DS2	Slight/minor	Malfunction of tank farm for a short time (less than 3 days) due to loss of backup power or light damage to tanks
DS3	Moderate	Malfunction of tank farm for a week or so due to loss of backup power, extensive damage to various equipment, or considerable damage to tanks
DS4	Extensive	Extensive damage to tanks or elevated pipes
DS5	Complete	Complete failure of all elevated pipes, or collapse of tanks

As an alternative, the HAZUS methodology proposes to consider not only the physical tank body (NIBS 2004), but the whole system needed to deliver the contents to the pipeline network: tank body, elevated pipes, commercial power, backup generators, electrical/mechanical equipment... Accounting for the role of each component, a damage scale that can be somewhat linked to functionality/ serviceability indicators is presented in Table 7.8.

7.4.2.3 Processing Plants/Stations

If the damage to support facilities is simply addressed by considering the vulnerability of the building that shelter them, then the associated damage scale is just the same as the one of the corresponding building (e.g. see Ghobarah 2004; Rossetto and Elnashai 2003; NIBS 2004).

On the other hand, if the emphasis is put on the mechanical or electrical components within the facility, then a global damage scale has to be derived for the local damage and functionality loss of the components, using a fault-tree analysis for instance. For example, a slight/minor damage (e.g. short-time malfunction of the plant) to the station may be induced by the loss of electrical power and backup generators, or a slight damage to the building. Such an approach was used in the LESSLOSS (2007) and SRMLIFE (2003–2007) projects, which resulted in the damage scale presented in Table 7.9.

It is to be noticed that the damage states described in Table 7.6 are not solely linked to the physical damage and that they were also defined so that they match the corresponding functionality loss.

Finally, regarding in-line valves or SCADA equipment, as it was explained before, no quantitative study of their vulnerability is available and therefore no relevant damage states can be defined for these components.

7.4.3 Description of Existing Functions

A critical review of existing fragility functions is proposed, based on the derivation method, the data used and the typologies covered.

Table 7.9 Damage scale proposed in LESSLOSS (2007) and SRMLIFE (2003–2007) for pumping/compression stations

Damage state		Damage description	Functionality loss	
DS1	None	No damage	Normal function	Full function
DS2	Slight/minor	Slight damage to building or full loss of commercial power and backup power for few days (<3 days)	Several stops and reduced flow of gas in the transmission gas pipelines	
DS3	Moderate	Considerable damage to mechanical and electrical equipment or considerable damage to building or loss of electric power and of backup for 7 days	Disability of boosting gas in compression station	Malfunction (full function after repairs)
DS4	Extensive	Building being extensively damaged, or the pumps badly damaged beyond repair		Full loss of function (unrepairable damage)
DS5	Complete	Building collapsed		

7.4.3.1 Pipelines

In the case of fragility functions for transient ground motion, a literature review has led to a non-exhaustive list of around 20 empirical relations, which are summed up in Table 7.10, along with the typologies they cover, the amount of data they rely on and the intensity measure they use. Some of these empirical relations are on a standard functional form (i.e. "backbone curve") and the differences in terms of diameter size, material and connection type are accounted for by the use of a multiplicative factor K that alters the final repair rate. Therefore the backbone curve of the fragility relation represents the repair rate under usual conditions (i.e. $K = 1$), while a smaller or greater K factor represents configurations where the resulting damage is reduced or amplified, respectively.

The relations described in Table 7.9 are all based on empirical data collected from post-earthquake observations. Usually, some adjustments to the raw data are performed: for instance, in the ALA (2001) methodology, only the damage to the main pipe is used to assess the relative vulnerability of different pipe materials. Also, data points assumed to contain permanent ground displacement effects can be eliminated when studying only the effects of ground shaking. Then, based on the data points, a correlation procedure is performed in order to fit a predefined functional form with the empirical data. For example, ALA (2001) explored a linear model ($RR = a.IM$) and a power model ($RR = b.IM^c$). Depending on the consistency of the available data, it is possible to build specific models based on various factors such as pipe material, pipe diameter or pipe connections.

Regarding the effects of ground failure, some specific fragility functions have also been empirically derived and they are detailed in Table 7.11.

Table 7.10 Summary of the existing pipeline fragility relations for transient ground motion

Reference	Typology	IM	No of earthquakes studied
Katayama et al. (1975)	Mainly cast-iron pipes Poor, average or good conditions	PGA	6
Isoyama and Katayama (1982)	Mainly cast-iron pipes	PGA	1
Eguchi (1983)	WSGWJ (welded-steel gas-welded joints), WSAWJ (welded-steel arc-welded joints), AC (asbestos cement), WSCJ (welded-steel caulked joints), CI (cast iron)	MMI	4
Barenberg (1988)	Mainly cast-iron pipes	PGV	3
Eguchi (1991)	WSGWJ (welded-steel gas-welded joints), WSAWJ (welded-steel arc-welded joints), AC (asbestos cement), WSCJ (welded-steel caulked joints), CI (cast iron), DI (ductile iron), PVC, PE (polyethylene)	MMI	4
O'Rourke et al. (1991)	–	MMI	7
Hamada (1991)	–	PGA	2
O'Rourke and Ayala (1993)	Brittle or flexible pipes	PGV	6
HAZUS (NIBS 2004)			
Eidinger et al. (1995)	Material type Joint type	PGV	7
Eidinger (1998)	Diameter Soil type		
O'Rourke et al. (1998)	Mainly cast-iron pipes	PGV, PGA, MMI	4
Isoyama et al. (1998)	Material type Diameter	PGV	1
Toprak (1998)	No distinction	PGV	1
O'Rourke and Jeon (1999)	Mainly cast-iron pipes Diameter	PGV	1
Eidinger and Avila (1999)	Material type Joint type Diameter Soil type	PGV	–
Isoyama et al. (2000)	Material type: CI, DI, PVC, steel, AC Diameter Soil type	PGA, PGV	1
ALA (2001)	Material type Joint type Diameter Soil type	PGV	18

(continued)

Table 7.10 (continued)

Reference	Typology	IM	No of earthquakes studied
Chen et al. (2002)	Diameter Material type	PGA, PGV, MMI	1
Pineda and Ordaz (2003)	Mainly brittle pipes (CI, AC)	PGV	1
O'Rourke and Deyoe (2004)	Mainly cast-iron pipes	PGV, PGS	5
Pineda and Ordaz (2007)	Mainly brittle pipes (CI, AC)	PGV^2/PGA	1
Maruyama and Yamazaki (2010)	Material type (CI, DI, PVC)	PGV	4
O'Rourke et al. (2012)	Mainly brittle pipes (CI, AC)	GMPGV (geometric mean of PGV)	4 (1 earthquake sequence)

Table 7.11 Summary of the existing pipeline fragility relations for ground failure

Reference	Typology	IM
Eguchi et al. (1983)	Material type: WS, AC, CI Joint type: gas-welded joints, arc-welded joints, caulked joints	PGD
Honegger and Eguchi (1992) HAZUS (NIBS 2004)	Ductile (DI, steel, PVC) or brittle (AC, concrete, CI) pipes	PGD
Heubach (1995)	Material type Joint type	PGD
Ballantyne et al. (1996)	Material type:	PGD
Eidinger and Avila (1999)	Ductile or brittle pipes	PGD
ALA (2001)	Material type Joint type	PGD
O'Rourke et al. (2012)	Material type: AC, CI, PVC	Angular distortion (β) and lateral strain (ε_{HP})

Comparatively, there are fewer examples of analytical fragility functions for buried pipelines. For instance, Terzi et al. (2007) developed fragility curves for the case of segmented pipelines subjected to permanent ground deformation, using a FEM model and accounting for pipe-soil interaction. The results were confronted with the case of a PVC pipeline that suffered damage from the 2003 Lefkas earthquake.

7.4.3.2 Storage Tanks

O'Rourke and So (2000) proposed empirical fragility curves for on-grade steel tanks based on more than 400 tank damages from 9 earthquake events in the United States (California and Alaska). The size of the available data allowed the authors to investigate the effects of two parameters, i.e. the tank's height-to-diameter ratio and the relative amount of stored contents. However, no distinction was made between anchored and unanchored tanks. A logistic regression enabled to fit the empirical data and to express the fragility parameters using a lognormal cumulative distribution.

The fragility curves proposed by ALA (2001) are also based on empirical data from 532 tanks, which experienced strong ground motions of 0.1 g or higher. A typology distinction is made depending on the percentage of stored contents and the anchorage of the tank to the baseplate. Like O'Rourke and So (2000), the ALA study concludes that the tanks that are less than half-full did not experience enough damage to compute fragility curves for DS4 and DS5. Thus, only the tanks with a fill percentage higher than 50 % were considered to estimate additional curves, based on the anchorage of tanks.

Based on the field observations previously reported by ALA (2001), Berahman and Behnamfar (2007) used a Bayesian approach to improve the empirical procedure. They accounted for both aleatory and epistemic (model bias, small data sample, measurement errors…) uncertainties. Fragility models are developed using a probabilistic limit state function and a reliability integral, solved with Monte-Carlo simulation. It was found that the fragility curves were less conservative than purely empirical models from ALA (2001), suggesting a better tank performance than expected. Also, one important result is that commonly-used lognormal distributions do not seem to be the best fit to the available empirical data. However, this study was only conducted for a specific typology of tanks (unanchored at-grade steel tanks) and other sets of fragility curves should be built to cover all typologies. Finally, the proposed fragility curves are based on an integral formulation and are not associated with an analytical form (like the lognormal distribution, which can be easily described with two parameters).

On the other hand, Iervolino et al. (2004) introduced an analytical approach to build fragility curves for unanchored steel tanks. Only one damage state is accounted for, i.e. failure by elephant's foot buckling. The final fragility curve is expressed as a cumulative lognormal distribution, which median and standard deviation parameters are evaluated through a response surface based on two variables (i.e. the fluid height-over-radius ratio and friction coefficient between the tank and the baseplate).

The study by Salzano et al. (2003) focuses also on the role of the fill level (i.e. near full or >50 %) for atmospheric tanks, with anchored or unanchored components. The results are based on empirical data from mostly north-American earthquakes.

Finally, the HAZUS methodology (NIBS 2004) proposes fragility curves for "tanks farms", accounting also for the fragility of the equipment that is needed for a proper operation of the tank (i.e. electric power, tank body, elevated pipes and various electrical/mechanical components). A fault-tree analysis is then used to assess the global functionality of the "tank farm" based on the specific damage state of each of its components.

7.4.3.3 Processing Plants/Stations

Various past research projects have tackled the issue of gas compression stations. For instance, a study from the European LESSLOSS project (LESSLOSS 2007) has proposed a hybrid approach, by considering both the fragility of the building and the logic tree relation between the components of the stations. A cumulative lognormal distribution of the damage probability was estimated, by using fragility curves by Kappos et al. (2006) for RC low-rise buildings with anchored components, designed with a low-level or advanced seismic code.

The SRMLIFE Greek project (SRMLIFE 2003–2007) also used this hybrid approach for pumping/compression stations, based on fragility curves for buildings by Kappos et al. (2006). The SRMLIFE study focused on specific fragility curves for Greek typologies (i.e. RC low-rise buildings with anchored components), while the fragility of the sub-components has been taken from the HAZUS methodology.

Finally, fragility curves have been also proposed in the HAZUS methodology (NIBS 2004), where the fragility curves of all components are used into a fault-tree analysis to obtain the global fragility function of pumping plants. A typological distinction is made between plants with anchored or unanchored components.

7.4.4 Comparative Analysis and Limitations

Based on the available fragility functions in the literature, a critical review is performed, with the aim of identifying the most adequate functions and the existing gaps.

7.4.4.1 Pipelines

According to the available typologies for gas & oil pipelines in Europe (mostly welded-steel, PVC and HDPE continuous ductile pipes), it is necessary to focus on fragility relations that are most adequate for ductile pipelines. Moreover, if we assume the use of PGV and PGD as respective intensity indexes for ground shaking and ground failures, it could be concluded that the following relations constitute good candidates:

- For wave propagation:
  - O'Rourke and Ayala (1993), which is used in HAZUS;
  - Eidinger et al. (1995) and Eidinger (1998);
  - Isoyama et al. (2000);
  - ALA (2001);
- For ground failure:
  - Eguchi et al. (1983);
  - Honegger and Eguchi (1992), which is used in HAZUS;
  - Eidinger and Avila (1999);
  - ALA (2001);

Some of these relations have been tested and confronted to a European case study (2003 Lefkas earthquake, Pitilakis et al. 2006), however only for water distribution pipelines (mainly brittle pipes): therefore these results may not apply to the specific case of gas and oil pipelines. More recently, Esposito et al. (2013) have compared available fragility curves in the literature with actual damage observations on the L'Aquila gas system.

Regarding the effects of transient ground motion, it is noted that the ALA (2001) study is the most recent one, as the HAZUS curves are still based on the O'Rourke and Ayala (1993) study. The ALA (2001) relations are based on the largest set of empirical data, including the 1994 Northridge earthquake: 18 events are used, as opposed to 6 in the study by O'Rourke and Ayala (1993). Moreover, the data from ALA (2001) is based on the study from O'Rourke and Ayala (1993) enriched with other datasets. In the ALA (2001) study, a balanced sample of U.S., Central American and Japanese earthquake is used, accounting for the variability of pipeline codes among various countries. Also, the consequent amount of data points (81, as opposed to 11 in O'Rourke and Ayala 1993) allows for a more balanced distribution of pipeline typologies.

Moreover, the review by Tromans (2004) compares some of the existing empirical relations: these curves are plotted on Fig. 7.6, assuming a corrective factor $K = 1$ (i.e. the "backbone curve", see Sect. 7.4.3.1).

As stated by O'Rourke (1999), the fragility relation by O'Rourke and Ayala (1993) seems to be over-conservative, with pipeline repair rates being unduly affected by the long durations of ground shaking experienced during the 1985 Michoacan earthquake (Tromans 2004). The relations by ALA (2001) and Isoyama et al. (2000) offer the longest applicability range, as opposed to the O'Rourke and Ayala (1993) and Eidinger et al. (1995) relations, which should not be extrapolated to large values of PGV. The use of the relations by O'Rourke and Ayala (1993) is also advocated by some validation studies carried out on the 1999 Düzce and 2003 Lefkas earthquakes, in the case of ductile pipelines (Alexoudi 2005). Finally, it may be useful to quote some of the conclusions drawn by Tromans (2004) in his review:

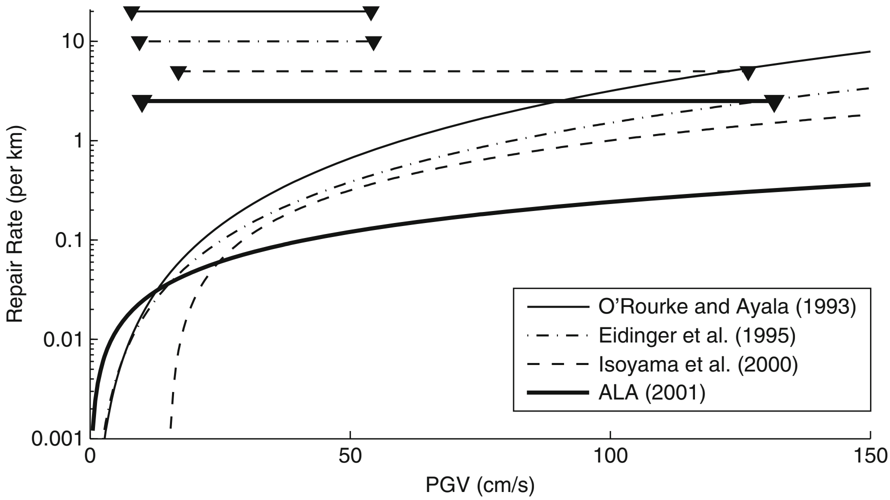

Fig. 7.6 Comparison of the pipeline fragility relations for PGV. *Straight lines* refer to the range of applicability of a given relation, approximated from knowledge of the dataset from which it was derived (Tromans 2004)

- The relation by O'Rourke et al. (1998) is to be used specifically in the U.S., as data from other locations have not been included: moreover, this relation should only be applied to cast-iron pipes.
- The relation by Isoyama et al. (2000) is suggested for Japan. Application to other locations is difficult, due to the specific topographic classification scheme, which is not normally used outside of Japan;
- For general applications, the relation by ALA (2001) is recommended, as it is derived from a global database.

The ALA (2001) relation may then represent an adequate solution to assess the vulnerability of buried ductile pipelines. It yields the repair rate (*RR* in repairs per km) as a function of PGV (in cm/s) via the following equation:

$$RR = K_1 0.002416 PGV \tag{7.3}$$

The parameter K_1 is used to adjust the fragility with respect to the backbone curve, based on the material, the connection type, the soil type and the pipe diameter (see Table 7.12).

The repair rate relation presented above allows assessing most of the specific typologies identified in the gas systems studied within the SYNER-G project (see Sect. 7.2.1):

- Greek transmission lines (WS, small diameter): $K_1 = 0.6$
- Greek distribution lines, Austrian transmission and distribution lines (PVC, small diameter): $K_1 = 0.5$
- Austrian supra-regional lines (WS, large diameter): $K_1 = 0.15$

Table 7.12 Some values of the K_I parameter in the ALA (2001) relation for transient ground motion, for welded steel and PVC pipes

Material	Joint type	Soil type	Diameter	K_I
WS	Arc welded	Unknown	Small	0.6
	Arc welded	Corrosive	Small	0.9
	Arc welded	Non corrosive	Small	0.3
	Arc welded	All	Large	0.15
	Rubber gasket	Unknown	Small	0.7
	Screwed	All	Small	1.3
	Riveted	All	Small	1.3
PVC	Rubber gasket	All	Small	0.5

A *small* diameter is considered to be comprised between 10.16 and 30.48 cm and a *large* one is greater than 40.64 cm

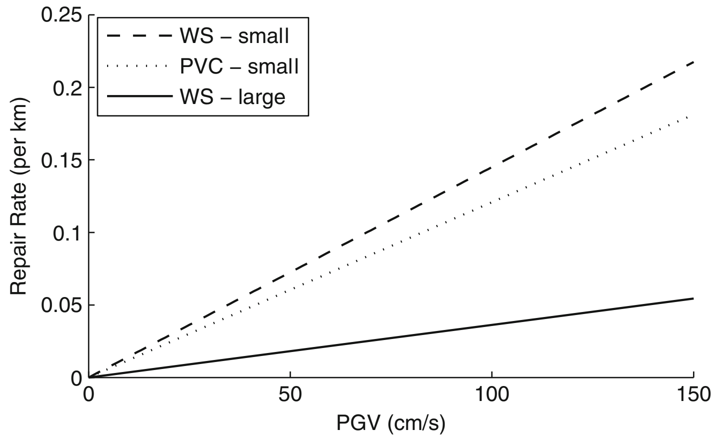

Fig. 7.7 Proposed repair rate for the most common gas pipeline typologies, due to wave propagation (ALA 2001)

The repair rate of these different configurations is plotted in Fig. 7.7 as a function of PGV.

However, existing relations fail to address the case of polyethylene pipelines (MDPE and HDPE), which are more and more commonly used in the gas distribution networks (e.g. the L'Aquila gas network). Still, the absence of observed damages on these pipes (O'Rourke et al. 2012, on the Canterbury earthquake sequence) leads to assume a very good response of these pipelines to seismic action.

Regarding the effects of ground failure, the relation by ALA (2001) is also the most recent one, as the one proposed by the HAZUS methodology is taken from Honegger and Eguchi (1992). The dataset from ALA (2001) comprises 41 data points from 4 earthquakes (one Japanese and three U.S.), with liquefaction as the main failure mechanism. Thus, the ALA (2001) curve is based on the most complete empirical data. A comparison of some fragility curves (use of the "backbone curve", without any corrective factors) is given in Fig. 7.8.

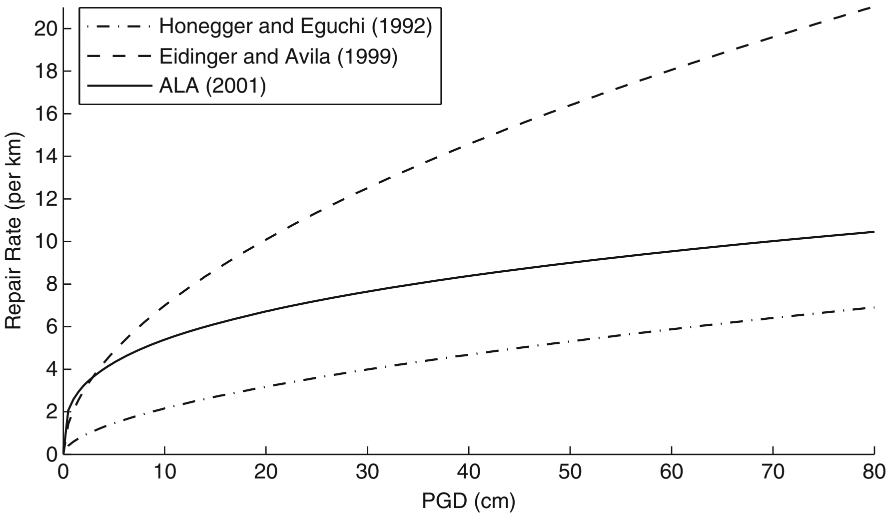

Fig. 7.8 Comparison of three repair rate relations for ground failure, with respect to PGD

Table 7.13 Some values of the K_2 parameter in the ALA (2001) relation for ground failure, for welded steel and PVC pipes

Material	Joint type	K_1
Unknown	Unknown	1.0
WS	Arc welded, lap welds	0.15
	Rubber gasket	0.7
PVC	Rubber gasket	0.8

Figure 7.8 shows important discrepancies between the different studies: the curve by (ALA 2001) lies in between the relations from Honegger and Eguchi (1992) and Eidinger and Avila (1999). Based on this discussion and in order to be coherent with the fragility curve selected for transient ground motion, we finally propose to adopt the relation from ALA (2001), as a function of PGD in cm:

$$RR = K_2 2.5829 PGD^{0.319} \tag{7.4}$$

The corrective factor K_2 depends on the pipe material and the connection type (see Table 7.13) and the following values are proposed in ALA (2001) for the most common pipelines typologies that are encountered in a European context (see Fig. 7.9):

- Greek transmission lines (WS, small diameter): $K_2 = 0.7$
- Greek distribution lines, Austrian transmission and distribution lines (PVC, small diameter): $K_2 = 0.8$
- Austrian supra-regional lines (WS, large diameter): $K_2 = 0.15$

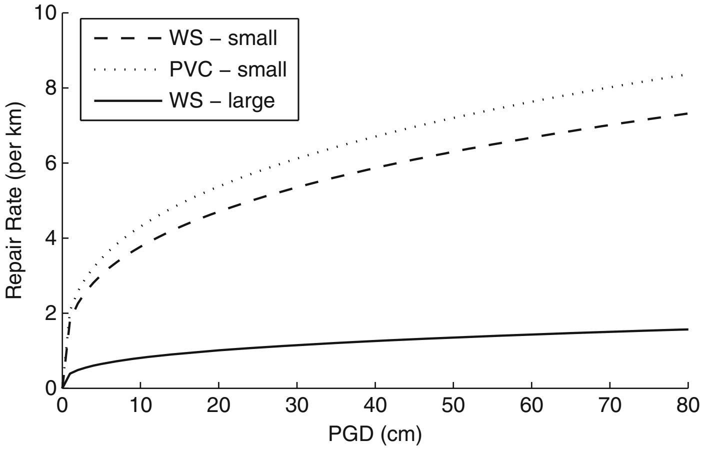

Fig. 7.9 Proposed repair rate for the most common gas pipeline typologies, due to ground failure (ALA 2001)

Again, the existing empirical relations fail to address the case of MDPE or HDPE pipelines. However, a recent experiment performed by O'Rourke et al. (2012) on a HDPE pipeline segment has revealed that the maximum strain (i.e. 8 %) induced by a strike-slip displacement of 1.2 m was far below the strain levels causing pipe wall rupture. However, the squeeze-off of the pipe and the associated loss of cross-sectional area were found to be a potential failure mechanism for polyethylene pipelines.

7.4.4.2 Storage Tanks

The studies by O'Rourke and So (2000) and ALA (2001) are the most thorough, as they allow for distinction between many characteristics such as the percentage of content stored, anchorage of components and height-over-radius ratio. However, some of the proposed fragility curves are based on really scarce empirical data, and this may raise issues on the reliability of the regression. Also, the damage states proposed by these two studies are mostly defined by physical damage mechanisms that prove to be difficult to link to any loss of functionality. Besides, oil storage tanks are located in very complex facilities (e.g. refineries, storage facilities...) and considering only the damage to the tank body seems to be a quite simplistic and rather non conservative approach: indeed, the whole "tank farm" system should be accounted for, including elevated pipes, power sources, mechanical equipment, etc.

It is then proposed to adopt the fragility curves for "tank farms" developed in the HAZUS methodology (NIBS 2004). These curves can be applied to on-grade steel tanks, with a distinction on whether components are anchored or not (see Table 7.14 and Fig. 7.10).

Table 7.14 Fragility parameters (median α and standard-deviation β) for tank farms proposed by HAZUS (NIBS 2004)

Typology	Damage state	α (g)	β
Tank farm with anchored components	Slight/minor	0.29	0.55
	Moderate	0.50	0.55
	Extensive		
	Complete	0.87	0.50
Tank farm with unanchored components	Slight/minor	0.12	0.55
	Moderate	0.23	0.55
	Extensive	0.41	0.55
	Complete	0.68	0.55

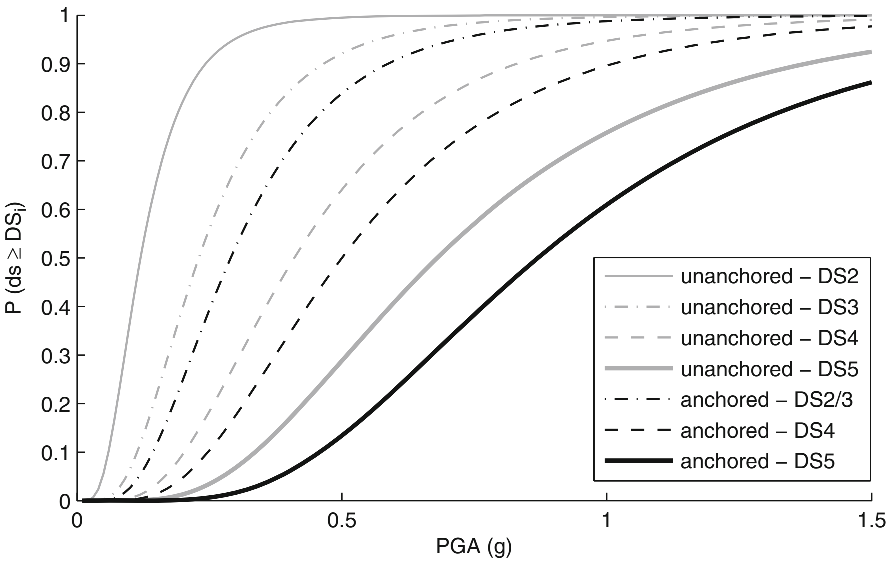

Fig. 7.10 Fragility curves for steel tank farms, proposed by HAZUS (NIBS 2004)

7.4.4.3 Processing Plants/Stations

The case of the Greek gas compression stations can be covered by the specific fragility functions that have been developed within the SRMLIFE Greek project. These fragility curves (see Table 7.15 and Fig. 7.11) are applicable to gas stations that are housed in low-rise RC buildings with anchored components.

Apart from the Greek context, the typology of generic European gas stations is not well known, and one solution could be to use the generic fragility curves of the HAZUS methodology (NIBS 2004), which are based only on the distinction between anchored and unanchored components (see Table 7.16 and Fig. 7.12).

In the case of the specific components identified in central Italy, there are no ready-to-use fragility functions. However, a fault-tree decomposition of the

Table 7.15 Fragility parameters (median α and standard-deviation β) for Greek compression plants, according to SRM-LIFE (2003–2007)

Typology	Damage state	α (g)	β
Anchored components, low-rise RC building (advanced code)	Minor	0.30	0.70
	Moderate	0.55	0.45
	Extensive	0.80	0.50
	Complete	2.20	0.70

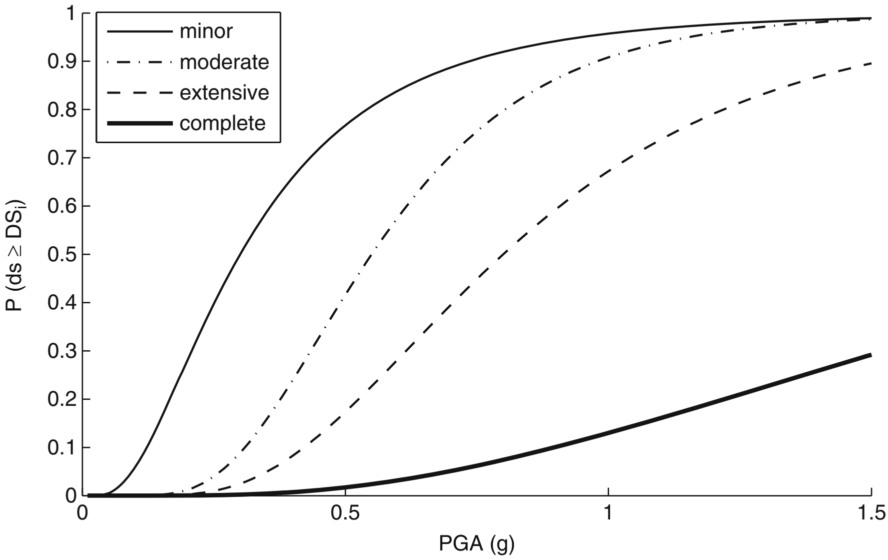

Fig. 7.11 Fragility curves for Greek compression stations, proposed by SRM-LIFE (2003–2007)

Table 7.16 Fragility parameters (median α and standard-deviation β) for pumping plants, according to HAZUS (NIBS 2004)

Typology	Damage state	α (g)	β
Anchored components	Minor	0.15	0.75
	Moderate	0.34	0.65
	Extensive	0.77	0.65
	Complete	1.50	0.80
Unanchored components	Minor	0.12	0.60
	Moderate	0.24	0.60
	Extensive	0.77	0.65
	Complete	1.55	0.80

sub-components may be helpful to assess the relative vulnerability of these stations. Regarding RE.MI cabins, all subcomponents are assumed to be unanchored and simply supported on the ground (with the exception of bowls that are located in a separated area and ceiling-mounted). These cabins may be decomposed in structural component (i.e. buildings), regulators and mechanical equipment (heat exchangers, boilers and bowls) and a fault-tree analysis is presented in Fig. 7.13.

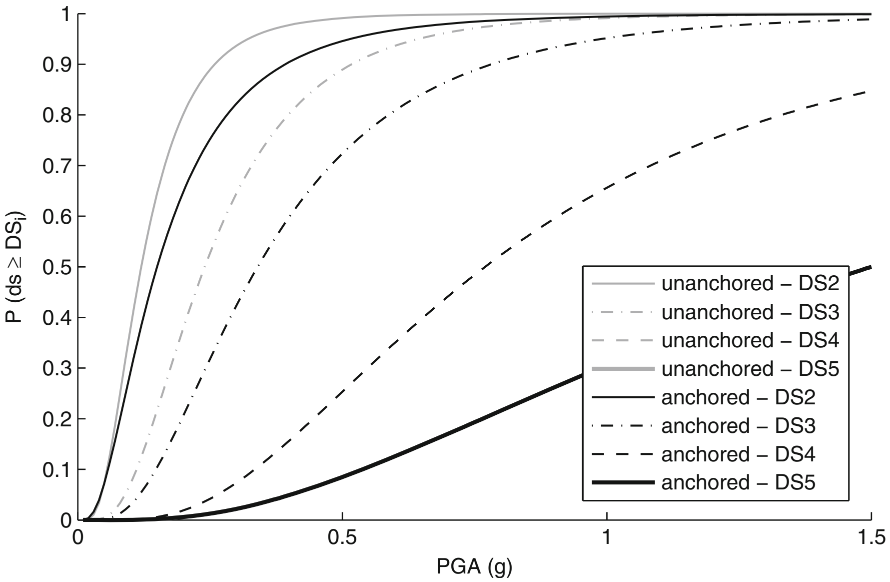

Fig. 7.12 Fragility curves for pumping plants, proposed by HAZUS (NIBS 2004)

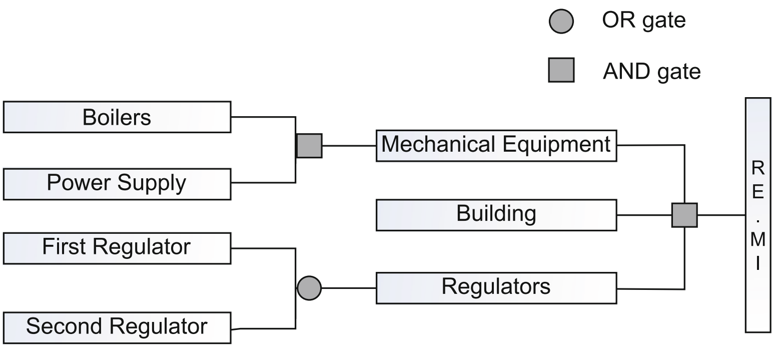

Fig. 7.13 Fault-tree decomposition of a RE.MI cabin (Esposito 2011)

Since gas supply has to be maintained at all times, two installations are mounted in parallel where each installation is characterized by a regulator and a monitor. The monitor is a safety device that has to be able to prevent the outlet pressure from exceeding safe thresholds in the case of complete failure of the regulator, taking over the function of the primary, normally active regulator. Besides, when boilers break down the gas flow is not ensured, since the freezing stops the system.

On the other hand, reduction groups (i.e. GR) can be broken down in regulators and masonry housing (when it is present, otherwise the group is sheltered within a kiosk) and the corresponding fault-tree is detailed in Fig. 7.14.

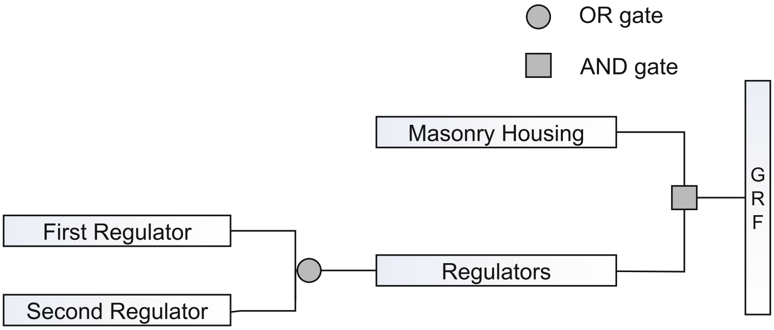

Fig. 7.14 Fault-tree decomposition of a reduction group (Esposito 2011)

In most cases the safety device is ensured by the presence of shut-off valves that are able to block the gas flow. When the pressure exceeds a maximum value, the valves close. However, some reduction groups do not include a second regulator and this characteristic implies a higher vulnerability.

7.5 Conclusions

The present review of available fragility functions for components of gas and oil networks has yielded some valuable lessons for future work on this topic. First, on a more general note, it appears that the case of the vulnerability of gas and oil networks should be more investigated regarding the dramatic consequences that can potentially result from component failures: most of the fragility functions presented here (especially for pipelines) have been developed for water supply networks and their adaptation to the case of gas and oil networks should be taken with extreme care. For instance, the impact of the different constitutive properties (especially the viscosity) of liquefied gas or oil should be investigated, thus introducing even more complex fluid–solid interactions in the analyses. The difficulty to properly tackle most of the modelling issues presented here may also be one of the reasons why most of the fragility functions are derived from empirical data and not from numerical computations or experimentations. The direct consequence is that these empirical relations, which are usually based on data from American or Japanese earthquakes, may not be suitable to European typologies.

In the case of buried pipelines, the emphasis has been put on ductile pipes, which are most common for gas and oil networks as opposed to brittle pipes (e.g. cast-iron, concrete) that are usually found in water supply systems. Whether damage is induced by transient wave propagation or by permanent ground deformation, it has been found that the use of the empirical relations by ALA (2001) might be reasonable for European pipeline typologies, which are essentially composed of welded-steel and PVC materials. For the networks where polyethylene pipelines are present, there is currently no fragility function available; however O'Rourke et al. (2012) confirm the excellent behaviour of this material under seismic action, since almost no damage has been recorded for MDPE or HDPE pipes in recent earthquakes.

A series of empirical fragility functions have also been reviewed, especially for at-grade steel tanks, which are common features of the vertical cylinders that are used to store hydrocarbons in oil refineries. The complexity of the mechanical and electrical equipment that is supporting the storage operations is accounted for by the HAZUS (NIBS 2004) fragility curves for 'tank farms'. However, the case of gas storage is less straightforward and the very specific features of the different storage facilities (e.g. LNG tanks for liquefied natural gas, underground cavities for seasonal storage, air-tight spherical or cylindrical tanks for special gases) prevent the use of generic fragility curves.

Finally, regarding processing facilities (i.e. compression or reduction stations), different levels of analysis are available, the most basic one consisting of the sole fragility analysis of the building housing the station. An alternative resides in the use of a fault-tree analysis to derive the global station fragility from the particular fragility of each of its subcomponents. Specific fragility curves are available for Greek gas compression stations and other compression/pumping stations could be assessed with generic fragility curves from HAZUS (NIBS 2004). More specific typologies have also been identified (i.e. RE.MI cabins and GRF groups in Central Italy) and, unfortunately, they could not be satisfyingly associated with any fragility functions. However, these stations could be decomposed into a fault-tree, revealing precious information on the criticality of some subcomponents and the relative vulnerability of stations that comprise redundant equipment or not.

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