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Abstract

This deliverable provides the technical report on the assessment of fragility functions for harbor elements. This deliverable comprises four parts. A short review of past earthquake damages on harbor elements is provided in the first part, including the description of physical damages, the identification of main causes of damage and the classification of failure modes. The following two parts deal with the identification of the main typologies or port components and the general description of existing methodologies, damage states definitions, intensity indexes and performance indicators of the elements. The validation of some empirical methods for waterfront structures is provided based on damage data from recent European earthquakes. Finally, improved vulnerability functions for the individual components are proposed along with the coding and digital description of fragility functions.

Keywords: fragility functions, vulnerability, harbor, port facilities, waterfront structures, cargo handling equipment

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1 Introduction

Port transportation systems are vital lifelines whose primary function is to transport cargos and people. They contain a wide variety of facilities for passenger operations and transport, cargo handling and storage, rail and road transport of facility users and cargoes, communication, guidance, maintenance, administration, utilities, and various supporting operations. Ports offer wide-open areas that can be used for emergency or refuge activities after a damaging earthquake. Moreover, ports can play an important role during the recovery period, as they can contribute to the reconstruction assistance and the transportation of goods for homeless citizens.

The combination of hazard, importance, vulnerability and exposure of the port structures, leads to a possibly high seismic risk. In fact, the consequences of earthquake-induced damage are not only related to life safety and repair costs of the structures, but especially to interruption of port serviceability in the immediate aftermath of an earthquake. Experience gained from recent seismic events (e.g. 1989 Loma Prieta in USA, 1995 Hyogoken-Nanbu and 2003 Tokachi-Oki in Japan) has dramatically demonstrated the seismic vulnerability of port structures and the severe damages that can be caused by ground shaking, as well as the potential economic losses due to earthquake damage.

In the framework of Work Package 3 – Fragility functions of elements at risk – and Task 3.3 (Fragility of elements within transportation infrastructures), the aim of this deliverable is to discuss and propose fragility curves and improved methods to assess the seismic vulnerability of harbor elements, applicable to the specific European context and are intended to be integrated to the general evaluation of the systemic vulnerability.

The present report reviews the damages sustained by harbor elements during past earthquakes, with special emphasis to European earthquakes. Different failure modes are identified and classified respectively. The following components are proposed to be studied within SYNER-G:

- Waterfront structures
- Cargo handling and storage components
- Infrastructures
 - Buildings (sheds and warehouses, office buildings, maintenance buildings, passenger terminals, traffic control buildings),
 - Utility systems (electric power system, water system, waste-water system, natural gas system, liquid fuel system, communications system, fire-fighting system),
 - Transportation infrastructures (roadway system, railway system, bridges).

The description of the European typology for the different components is performed. A review of existing methodologies for the vulnerability assessment of harbor elements is followed by the definition, for each component, of some key parameters:

- Damage states scales.
- Intensity index (indices) (intensity-measure parameter).

- Performance indicators that can help specify the link between the damage state of the component and its serviceability / functionality.

Finally, based on the review of state-of-the-art fragility curves for each component, and the validation of some methods based on damage data from recent European earthquakes, improved fragility functions for the individual components are proposed along with their coding and digital description. For the proposed vulnerability functions, the following parameters are provided:

- Typology classification of each component.
- Damage scale definition.
- Intensity index used.
- Fragility curve parameters, for each damage state and each typology.

2 Past earthquake damages on harbor elements

Ports may be prone to damage from earthquakes, not only under severe levels of shaking but under moderate levels as well. Furthermore, as shown by the experience of the Port of Oakland during the 1989 Loma Prieta Earthquake, the Port of Kobe during the 1995 Hyogo-ken Nanbu (Kobe) Earthquake, and many other port facilities during past earthquakes, seismic damage to ports can cause extended economic losses to the port, its stakeholders, and regional/national interests (Werner et al., 1999).

2.1 PHYSICAL DAMAGES / MAIN CAUSES OF DAMAGE

By far, the most significant source of earthquake-induced damage to port and harbor facilities is the increase of induced earth pressures caused by inertial forces to the retained ground mass and by hydrodynamic and pore-water pressure build-up in the saturated cohesionless soils that prevail at these facilities. This pressure build-up can lead to excessive lateral pressures to quay walls. Liquefaction and massive submarine sliding are also very important causes for spectacular failures (ATC-25). Other sources are local permanent ground displacements, ground failure and extensive settlement related with ground shaking (Pachakis and Kiremidjian, 2004; Werner, 1998). Yet, the liquefaction of loose, saturated, sandy soils that often prevail at coastal areas (especially reclaimed land and uncompacted fills) is the most widespread source of seismic damage to port structures. Past experience has demonstrated that even moderate levels of earthquake intensity can cause liquefaction, resulting in the reduced stiffness and loss of shear strength of the liquefied soils. This can in turn result in induced soil settlements, increased lateral earth pressures against retaining structures and loss of passive resistance against walls and anchors (PIANC, 2001). Finally, port structures are also subjected to damage due to tsunamis. Buildings in port areas are subject to damage due to shaking, as well as damage caused by loss of bearing or lateral movement of foundation soils.

There are a large number of references regarding seismic damage of port structures, mostly during earthquakes in the USA and Japan. Table 2.1 shortly summarizes the seismic response and induced total losses (direct and indirect) during strong earthquakes, and some recent earthquake events, from the international field.

Table 2.1 Seismic response of port facilities during strong earthquake events

Earthquake	Seismic response and induced damage of port facilities	Reference(s)
Loma Prieta U.S.A 17/10/1989, $M_s=7.1$	<p>Minor damages occurred to the Port of Redwood City (mainly failures of water pipelines and batter piles) without loss of functionality. The main failure in the Richmond port was the rupture of a fuel storage tank. To the ports of San Francisco and Oakland induced damages were mainly due to the liquefaction of embankments causing differential settlements of waterfront structures on embankments in relation to pile-supported structures. Numerous building failures and gas pipelines failures have been presented to the port of San Francisco. The most prominent losses have been sustained by the port of Oakland (cost ~ \$75 million). Damages occurred to cranes, water pipelines and tanks leading to hazardous materials spill-off.</p>	EERI, 1990 Ferritto, 1997
Hyogo-ken Nanbu (Kobe) Japan 17/1/1995, $M_w=6.9$	<p>Extensive damage occurred to the port of Kobe mainly due to liquefaction of embankments, increased lateral forces, settlements and lateral spreading. Pile-supported structures had better seismic response, in respect to structures on other types of foundation which sustained differential settlement, tilt and major failures. Lateral spreading due to liquefaction was up to 3m, subsidence and tilt of quay walls up to 3°, pipeline failures, and closure of the 179 out of the 186 piers. Tanks and cranes foundations also sustained damage. Extensive liquefaction occurred to the artificial Islands of Port and Rokko. In Rokko Island, the maximum soil settlement behind a quay wall was 3 m, while in some locations waterfront structures induced seaward sliding equal to 2 m. Deep ground ruptures occurred parallel to the waterfront. The majority of container cranes de-railed. One crane totally collapsed, while others sustained structural members damage. Finally, liquefaction induced damage occurred to other port facilities. More than 50% of pavements surface in Port Island was covered by sand boils.</p>	Bardet et al., 1995 Ferritto, 1997 PIANC, 2001
Chi-Chi Taiwan 21/9/1999, $M_s=7.6$	<p>Damage occurred to 4 out of the 85 piers in the port of Tai-Jung due to the occurrence of liquefaction phenomena, as well as in cargo storage areas and roads passing behind piers. Seaward sliding and tilting of caisson quaywalls (up to 3,5%) were observed, with maximum differential movements along the shoreline equal to 1,6m and maximum base sliding equal to 0,9m. Main attributed causes are the seismic inertia forces in combination with increased pressure of the retained soil due to liquefaction. Ground settlements (60-100cm) and lateral spreading (20-50cm) of embankments, and sand boils were observed. Damage also occurred to cranes, small buildings and pipelines in areas where liquefaction occurred, as well as to a tank due to ground shaking.</p>	EERI, 2001

Earthquake	Seismic response and induced damage of port facilities	Reference(s)
Costa Rica 22/4/1991, $M_w=7.7$	Extensive liquefaction occurred to the port of Limon with settlements up to 60cm in the areas built over fill embankments. The port functionality was reduced to 70% for a week after the earthquake. The operations for the full recovery of the activities lasted one month.	EERI, 1991
Hokkaido-Nansei-Oki Japan 12/7/1993, $M=7.8$	Ports of Esashi, Aonae and Okushiri-Cho were hit by tsunami. In some cases there were disruptions to transport of ferry boats and functions of shops and other port facilities due to debris and vehicles. Damage occurred to concrete quay walls (turnover and movement) due to tsunami and/ or geotechnical factors (increased lateral forces and soil liquefaction).	EERI, 1995
Bhuj (Gujarat) India 26/1/2001, $M_s=7.9$	Substantial losses occurred to the Kandla port, the largest port in India; damage occurred to wharfs, warehouses and operations buildings. Damage to the older wharfs is reported to consist of flexural and shear cracking at the tops of the reinforced concrete piles, possibly related to ground failure at the site. The newer wharfs, supported on larger diameter piles, and gantry cranes were undamaged. The port was closed for one week, with 20 days after the earthquake the port was operating at about 40-50% of capacity. Repair costs are estimated to amount to \$30 million. Significant damage also occurred at the Navlahki port. Lateral spreading to the south and subsidence caused over a kilometer of the main access road and railroad track to drop, with portions inundated during high tide. Damage was also induced to wharf structures and warehouses due to soil failure. The Mundra port, constructed one year prior to the earthquake with private assistance, was reported to have no significant damage.	Abrams et al., 2001
Fukuokaken - seiho-oki Japan 20/3/2005, $M_j=7.0$	Liquefaction occurred in many reclaimed lands and caused damages to quay walls. In Oki-hama, a quay wall tilted. Horizontal displacement of the quay wall was about 2 m. The ground behind the quay wall flow toward bay and subsided. Flowed zone extended about 10 m behind the quay wall.	Yasuda and Tanaka, 2010

On the other hand, records of damage sustained by waterfront structures are quite limited in European earthquakes. The Kocaeli earthquake (Turkey, 17/8/1999, $M_s=7.4$), caused extensive damage to the numerous waterfront piers and port facilities around Izmit Bay. Extensive damage was observed at the Golcuk Navy Base due to fault crossings. Damage included failure of piers, mechanical equipment, piping and the collapse of cranes. The government owned Derince general cargo and grain port, which handles some 2 million tons of cargo annually, suffered heavy damage to docks, cranes and warehouses, including cracks and severe subsidence while the port at Tuzla (west of Derince) experienced only minor damage and the port at Haydarpasa (close to Istanbul) was undamaged. Derince Port, in which the quay walls are of the concrete caisson type with shorefront length of about 1.5km, shifted away from the wharf up to 0.7m horizontally and 1m vertically due to

liquefaction-induced deformations, settlements and lateral spreading. Several rail mounted portal cranes and some old steel warehouses were damaged. A substantial number of the jetties at the industrial facilities were also damaged. These include Petkim facilities, Tupras Oil Refinery, Petrol Ofisi, Shell Oil, Trans Turk, Seka Paper Mill and UM Shipyard (Erdik, 2000). Some examples of damage are given in Fig. 2.1.

The military port at Gölcük sustained damage to waterfront structures due to ground failure and surface fault rupture (EERI, 2000).



Fig. 2.1 Damage to waterfront structures and supporting piles during the 1999 Kocaeli earthquake



Fig. 2.2 Damaged quay walls at Golcük navy base during the 1999 Kocaeli earthquake

In Greece, earthquake damage to waterfront structures has been recorded during the Kalamata (13/9/1986, $M_s=6.2$) and Lefkas (14/8/2003, $M_s=6.4$) earthquakes. The Kalamata main harbor quay wall, in spite of being damaged, preserved its serviceability during and after the earthquake. The quaywall displaced horizontally about $0,15\pm 0,05$ m and rotated $4-5^\circ$ toward the sea. The backfill presented a subsidence varying from 20 cm behind the wall to zero at a distance of 30 to 40 meters away. The backfill subsidence is due partially to the wall movements and partially to its own compaction that took place during the earthquake, as it was in a quite loose initial formation (Pitilakis and Moutsakis, 1989).

During the Lefkas earthquake, damage was observed to the quaywalls of the city port and Marina waterfront structures, mostly consisting of lateral sliding and tilting of walls, settlement of backfills and apron pavement cracking. Liquefaction was most probably the primary source of seismic damage in most cases. In the city of Lefkas, extensive horizontal movement (up to about 15cm) and tilting of the waterfront retaining structures were observed for hundreds of meters, as well as ground settlement - permanent vertical or differential displacement behind and along the quay walls (varying from 1 to 40 cm) and damage to pavement aprons, mostly due to partially of fully liquefied soils and backfill materials (Kakderi et al., 2006). Waterfront structures in Marina harbor experienced differential horizontal displacements up to 12 cm and settlements varying from 1-40cm. Seismic failures of various severity have also been sustained by numerous other port and harbor facilities in the island (Ligia, Nidri, Vasiliki). The most severe damage of the island waterfront structures were observed in Ligia Port, where the retaining structures collapsed along a significant part of the coastline.

2.1.1 Waterfront structures

Most failures of waterfront structures are associated with outward sliding, deformation and tilting. Extensive seismic damage is usually attributed to the occurrence of liquefaction phenomena. Residual tilting reduces the static factor of safety (FS) after the earthquake, while sliding is more a serviceability rather than a safety problem. There are a large number of references regarding seismic damage of port structures, mostly after earthquakes in the USA and Japan. In Europe similar observations are quite limited, while the majority of port structures in Europe are located to moderate to high hazard zones based on the national seismic codes (design PGA values) (Borg and Lai, 2007).

Experience demonstrates that waterfront structures are often prone to earthquake induced ground failures (Figs. 2.3, 2.4). Lateral ground displacements generated by liquefaction-induced lateral spread causes substantial damage to port structures as observed from past earthquakes such as the 1995 Hyogo-ken Nanbu (Kobe) (Fig. 2.5) and 2001 Nisqually, and often even under moderate earthquakes (Werner, 1998). It was found that the occurrence of the liquefaction in back fill was the main reason for the damage from many recent earthquakes to gravity quay walls such the 1964 at Nigata Port, the 1993 at Kushiro- oki and the 1994 at Hokkaido Toho- oki (Na et al., 2009a).

The seismic behaviour of port structures is largely governed by the local soil conditions. Past earthquakes demonstrated that the seismic behavior of port structures such as quay walls was significantly governed by the properties of soils. Actually, during the 1995 Hyogo-ken Nanbu (Kobe) earthquake, the caisson type quaywalls of identical configuration, located at the site in a close proximity with a similar average soil property, experienced significantly different degrees of damage. To show this variability of response of quaywalls, residual

horizontal displacements (RHD) of the caissons recorded through the survey after the 1995 Hyogo-ken Nanbu (Kobe) earthquake (Inatomi et al., 1997) are shown in Fig. 2.6(a). Fig. 2.6(b) depicts a typical section in the Rokko Island in Kobe port. Relatively poor seismic performance of many ports is primarily due to the poor foundation and backfill soils that are common in waterfront environments.



Fig. 2.3 Extremely extensive damage of apron pavements during the 1995 Hyogo-ken Nanbu (Kobe) earthquake

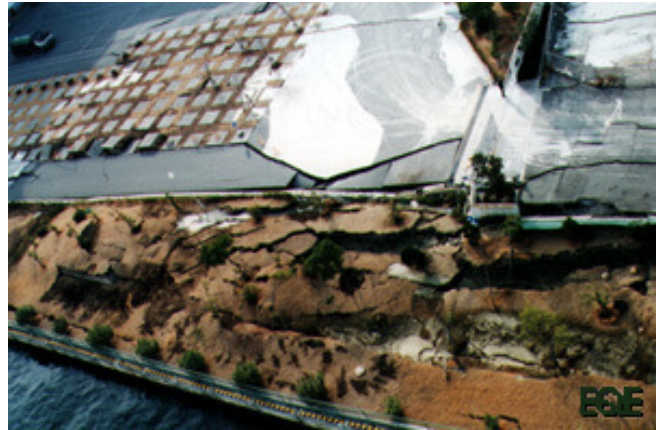


Fig. 2.4 Lateral spreading, liquefaction and settlement along the shore of the Port of Kobe during the 1995 Hyogo-ken Nanbu (Kobe) earthquake



Fig. 2.5 Seaward displacement and tilting of quay-walls during the 1995 Hyogo-ken Nanbu (Kobe) earthquake

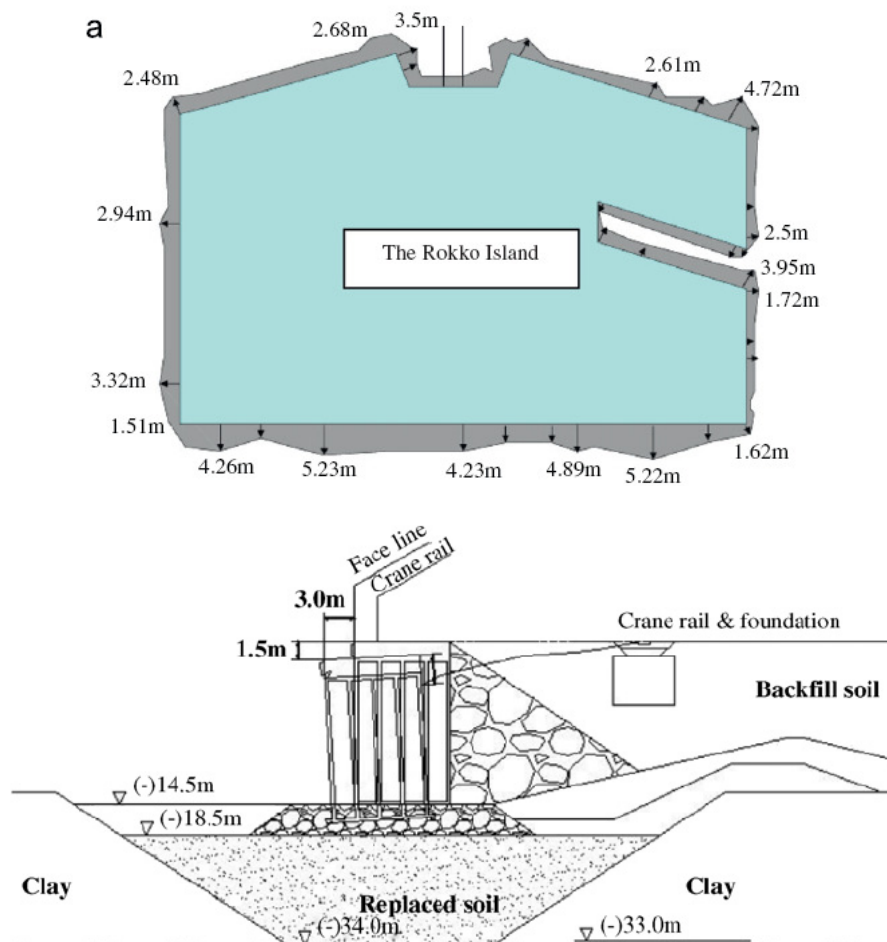


Fig. 2.6 (a) Field observation of RHD for damaged quay walls at the Rokko Island in Kobe port and (b) typical damaged section of a quay wall

2.1.2 Cargo handling and storage components

Experience from past earthquakes shows that properly designed cranes perform well if the foundations and soils perform well. Damages to cranes after earthquake events could be attributed not only to ground shaking, but also due to movement of rail foundation caused by ground failure, resulting in bending of their members (Figs. 2.7, 2.8). While not in use, cranes are restrained or anchored to foundation rails, preventing their relative movement or derailment. In this case, they are subjected to inertia forces, like any other structure with rigid foundation connections, rendering them vulnerable to failure due to bending and ground shaking. However, even in cases where relative movement or derailment is possible (for example when anchors have failed or while cranes are in use), cranes may overturn due to liquefaction of underlying soil fills or/and the occurrence of differential settlements, or they may be induced to bending type of failure due to ground detachment of a foundation member (PIANC, 2001). Overturned cranes may induce damage to adjacent structures and other facilities. Finally, disruption of cranes functionality may be induced by settlement and/or horizontal movement of foundation rails due to liquefaction of subjacent soil layers, as were the cases of the ports of Oakland, Derince and Kobe during the Loma Prieta, Kocaeli and Hyogo-ken Nanbu (Kobe) earthquakes (Figs 2.9, 2.10). Rail de-alignment may cause

damage to wheels and cranes immobilization (ATC-25). Downtime of cranes may vary from few days (in cases of simple derailment) up to several months due to severe earthquake damage.

Early container cranes are lighter and less stable than today's larger jumbo cranes. During an earthquake, the earlier cranes would lift from the rails before significant inertia forces could develop in the crane structure. Current large cranes, with 100-foot or greater rail gages, are much heavier, which results in significantly larger seismic forces in the crane structure. Recent studies (Soderberg et al., 2009) indicate that many jumbo cranes will be extensively damaged in moderate earthquakes, and that many jumbo cranes will be severely damaged, or will collapse, in a major earthquake.

Past seismic performance of tanks is generally poor. Supporting structures of other cargo handling and storage components are particularly susceptible to seismic damage according to change in material stiffness.



Fig. 2.7 Seaward lateral movement of rail foundation and differential settlement of rails [1995 Hyogo-ken Nanbu (Kobe) earthquake, Rokko Island]



Fig. 2.8 Crane total collapse. Various damages to other cranes like plastic hinges and bending of their members [1995 Hyogo-ken Nanbu (Kobe) earthquake, Rokko Island]



Fig. 2.9 Crane damage during the 1995 Hyogo-ken Nanbu (Kobe) earthquake



Fig. 2.10 Sand emersion near container cranes [1995 Hyogo-ken Nanbu (Kobe) earthquake, Port Island]

2.1.3 Port infrastructures

Port infrastructure components include in one hand utility and transportation systems, and in the other hand buildings. All utility systems may be present at ports. Both roadway and rail access links are commonly required to transfer waterborne cargo and people.

There are examples of both poor and good seismic performance of utility lifelines at ports during past earthquakes. The most common cause of seismic damage to roadway and railway systems is Permanent Ground Displacement (PGD).

Beyond common damage states, alternate expressions of losses seem relevant for infrastructure buildings:

- Functionality (yes or no);
- Serviceability
 - Nominal use, reduced use or not usable.
 - Usable without repairs, after repairs or not repairable.

In the framework of the European research project RISK-UE (2001-2004), the main modes of failures for the port infrastructures components have been identified and reported.

Traffic control building, lighthouses and beacons belong to the port systems. Their main functions are observation, control, navigation and communication. Observation/control offices, often with critical operational yard functions or vessel navigation/communication roles, usually occupy upper stories of a traffic control building. They may also contain important data cablings and radio communication links.

Modes of failures may be the following:

- Excessive tilting or permanent drift.
- Overturning.
- Racking deformations.
- Torsional deformations.

The building could also be prone to non-structural damage such as:

- Damage to staircases and elevators.
- Loss of function due to utility failures.

Passenger terminal is a building belonging to the port systems, serving as an important access point to the ships. By nature, human occupancy in passenger terminals could be high.

Modes of failures may be the following:

- Walls pulling away from roof diaphragms.
- Roof beams pulling away from wall or column supports.
- Column subsidence (and roof or wall damage).
- Floor spreading or settlement.
- Wall failures.
- Excessive deformations in roof trusses or tapered girders due to column failure or building racking.

The building could also be prone to non-structural damage such as:

- Glass window and door breakage.
- Damage to partitions, awnings, mezzanines, catwalks, gangways, concourses, staircases, and elevators.

Office buildings are constructions belonging to the port systems in which different tasks are carried out, like command centres, port management, administration, engineering and security functions. Office buildings at ports are usually extensively occupied and house critical operations.

In addition to usual building damage modes, port office buildings could face problems because of liquefiable soils:

- Damage to pile foundations.
- Settlement of foundations.

The building could also be prone to non-structural damage such as:

- Glass window and door damage.
- Inadequate anchorage or bracing of non-structural elements.

Maintenance buildings are constructions belonging to the port systems used for repairing equipment or keeping it in proper condition. Maintenance facilities are not usually critical for system functionality as maintenance activities can be delayed or performed elsewhere. They have significantly more personnel occupancy, utilities and mechanical equipment than shed and warehouse structures. They may house hazardous chemical and gas storage tanks, fuel systems, overhead cranes, high roof bays and material storage racks.

Modes of failures may be the following:

- Walls pulling away from roof diaphragms;
- Roof beams pulling away from wall or column supports;
- Column subsidence (and roof or wall damage);
- Floor spreading or settlement;
- Wall failures;
- Excessive deformations in roof trusses or tapered girders due to column failure or building racking;
- Foundation damage (differential settlement).

The building could also be prone to non-structural damage such as:

- Utility breaks;
- Hazardous spills or gas release;
- Fire;
- Equipment toppling;
- Material falling from overhead racks;
- Damage to under-slabs piping and tankage.

Sheds and warehouses are buildings belonging to the port systems, where goods are stored prior to their use, distribution, or sale. They may be distant from the shoreline, or located on the wharf itself. Sheds and warehouses generally have low human occupancy and contain a variety of cargoes ranging from bulk products to manufactured goods.

Modes of failures may be the following:

- Walls pulling away from roof diaphragms;
- Roof beams pulling away from wall or column supports;
- Column subsidence (and roof or wall damage);
- Floor spreading or settlement;
- Wall failures;
- Excessive deformations in roof trusses or tapered girders due to column failure or building racking.

2.2 CLASSIFICATION OF FAILURE MODES / DIRECT LOSSES

In general, damage to port facilities may be the following (RISK-UE, 2001-2004):

- Deformation and failure of dikes;
- Lateral sliding, rotation, settlement and deformation of retaining structures;
- Buckling, yielding and fracture of pile supports at piers and wharves (particularly battered piles);
- Displacement of crane rails with resulting disruption of crane operations;
- Extensive settlement and cracking of pavements within storage yards and along access roadways;
- Damage to buried or collocated pipelines;
- Potential for disruption of supporting lifelines that service port facilities.

2.2.1 Waterfront structures

Most failures of waterfront structures are associated with outward sliding, deformation and tilting of quay walls and sheet-pile bulkheads. Block-type quay walls are vulnerable to earthquake-induced sliding between layers of blocks. This damage has often been accompanied by extensive settlement and cracking of paved aprons. The principal failure mode of sheet-pile bulkheads has been insufficient anchor resistance, primarily because the anchors were installed at shallow depths, where backfill is most susceptible to a loss of strength due to pore-water pressure build-up and liquefaction. Insufficient distance between the anchor and the bulkhead wall can also lead to failure. In case of waterfront structures supported on piles, possible failure modes are also related to seismic damage induced to piles themselves. Pile-supported docks typically perform well, unless soil failures such as major submarine landslides occur. In such cases, piers have undergone extensive sliding and buckling and yielding of pile supports. Batter piles have damaged pier piles caps and decking because of their large lateral stiffness (ATC-25). Finally, increased damage

probabilities due to differential ground settlement are attributed to structures supported by different types of foundations (surface foundations on landfills and pile foundations).

After the 1995 Hyogo-ken Nanbu (Kobe) earthquake, the field investigation reports show that the characteristic damage patterns of a caisson-type quay wall are large lateral movements, tilting, and settlements of caissons and ground movements of the backfill in the form of lateral movements and settlements of the apron (Inatomi et al., 1997). In the case of caisson-type quay walls subjected to earthquake excitation, typical damage modes are found out to be seaward displacement, settlement and tilting. Fig. 2.11 shows a schematic drawing for the various damage modes of a typical caisson-type quay wall (Na et al., 2008). Note that the settlement of foundation and lateral spreading of backfill has also been shown in this figure.

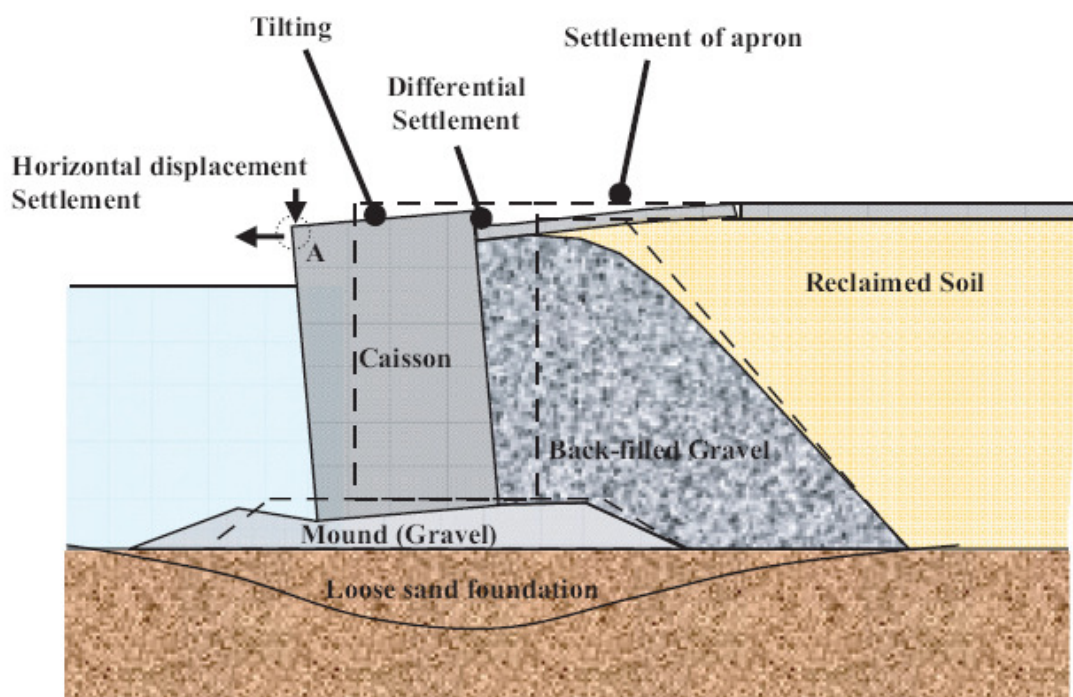


Fig. 2.11 Typical damage modes for gravity type quay walls and parameters for damage criteria

For gravity-type quay walls, possible modes of seismic failure are classified as indicated in Table 2.2 (Kakderi et al., 2006).

Table 2.2 Earthquake induced failure modes of gravity-type quay walls

Component	Failure modes
Quay walls	Outward sliding
	Tilting
	Settlement
	Overturning and extensive tilting
	Collapse
	Apron pavement cracking
Backfill materials	Cracking with corresponding pavement settlement relative to wall
	Ground fracture and cracking of road surface
	Waterspouts from ground fissures and sand boils
	Settlement of backfill
	Differential ground settlement
	Lateral ground movement (lateral spreading)

Representative types of earthquake induced damage to quay walls are illustrated in Fig. 2.12 to Fig. 2.16.

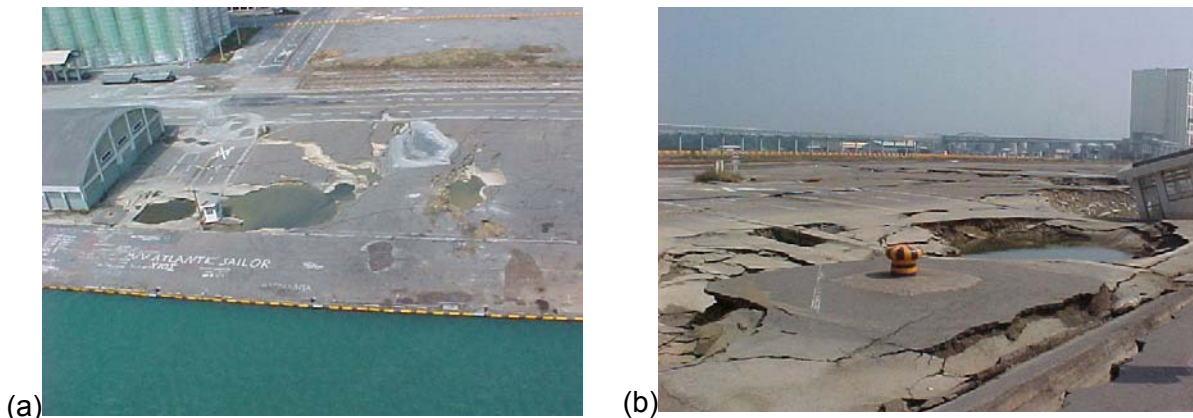


Fig. 2.12 Ground deformation due to liquefaction of apron pavements behind quay walls(a) and pavement damage (b) during the 1999 Chi-Chi earthquake



Fig. 2.13 Seaward movement of quaywall during the 203 Tokachi-Oki earthquake



Fig. 2.14 Turnover and extensive tilting of quay-walls during the 1995 Hyogo-ken Nanbu (Kobe) earthquake



Fig. 2.15 Backfill settlement behind quay wall and damage to apron pavement in the city's port during the 2003 Lefkas earthquake



Fig. 2.16 Backfill settlement (25cm), seaward sliding and tilting of quay walls in the city's Marina during the 2003 Lefkas earthquake

2.2.2 Cargo handling and storage components

Early cranes may be tipped and lifted from their rails without significant damage. Some cranes that tip may not land on their rails. Often, even though the wheels are off the rails, the portal frame is still elastic. Restoring the wheels to the rail is not difficult because after lifting the frame with jacks, the portal frame pulls the wheels back over the rail.

When crane supports fail, due to ground failure, significant damage may occur. If the crane rails spread, the crane legs spread until plastic hinging develops in the portal frame. Cranes may collapse due to excessive spreading.

The jumbo cranes are more massive, have greater portal clearances, and are more stable, resulting in much larger lateral loads. The strength of the portal frame has not kept up with the increased lateral seismic loading demand. If modern jumbo cranes are designed to early criteria, they cannot tip elastically and permanent damage will occur. For these cranes, the following performance is predicted (Soderberg et al., 2009):

- In a design operating level earthquake (OLE) of 72 year mean return interval (MRI), the portal frame would suffer significant damage consisting of localized plate buckling at the leg to portal connection and possibly other areas of the portal frame.
- In a large 475 year MRI earthquake, the portal frame would be significantly damaged resulting in possible crane collapse. Performance in this level earthquake is highly dependent on the ability of the plastic hinges to maintain adequate strength over the many cycles of loading.
- Damage to the portal frame occurs due to lateral displacement in the trolley travel direction.

3 Methodology for the vulnerability assessment of harbor elements

3.1 IDENTIFICATION OF THE MAIN TYPOLOGIES

Port transportation systems are vital lifelines whose primary function is to transport cargos and people. They contain a wide variety of facilities for passenger operations and transport, cargo handling and storage, rail and road transport of facility users and cargoes, communication, guidance, maintenance, administration, utilities, and various supporting operations.

The various components within a major port could be classified in the following categories (RISK-UE, 2001-2004; LESSLOSS, 2004-2007):

- Earthen embankments (hydraulic fills and native soil materials);
- Waterfront structures;
 - Retaining structures/dikes (e.g. at wharves, embankment, breakwaters, and dredged shipping lanes and waterway),
 - Berthing structures;
- Cargo handling and storage components;
 - Container storage areas,
 - Liquid storage tanks,
 - Material handling equipment;
- Infrastructure components;
 - Utility systems (electric power system, water system, waste-water system, natural gas, liquid fuel system, communications system, fire-fighting system),
 - Transportation infrastructures (roadway, railway, bridges),
 - Buildings.

In a port system, the following elements will be considered in SYNER-G (

Fig. 3.1):

- Waterfront structures
- Cargo handling and storage components
- Infrastructures
 - Buildings (sheds and warehouses, office buildings, maintenance buildings, passenger terminals, traffic control buildings),
 - Utility systems (electric power system, water system, waste-water system, natural gas system, liquid fuel system, communications system, fire-fighting system),

- Transportation infrastructures (roadway system, railway system, bridges).

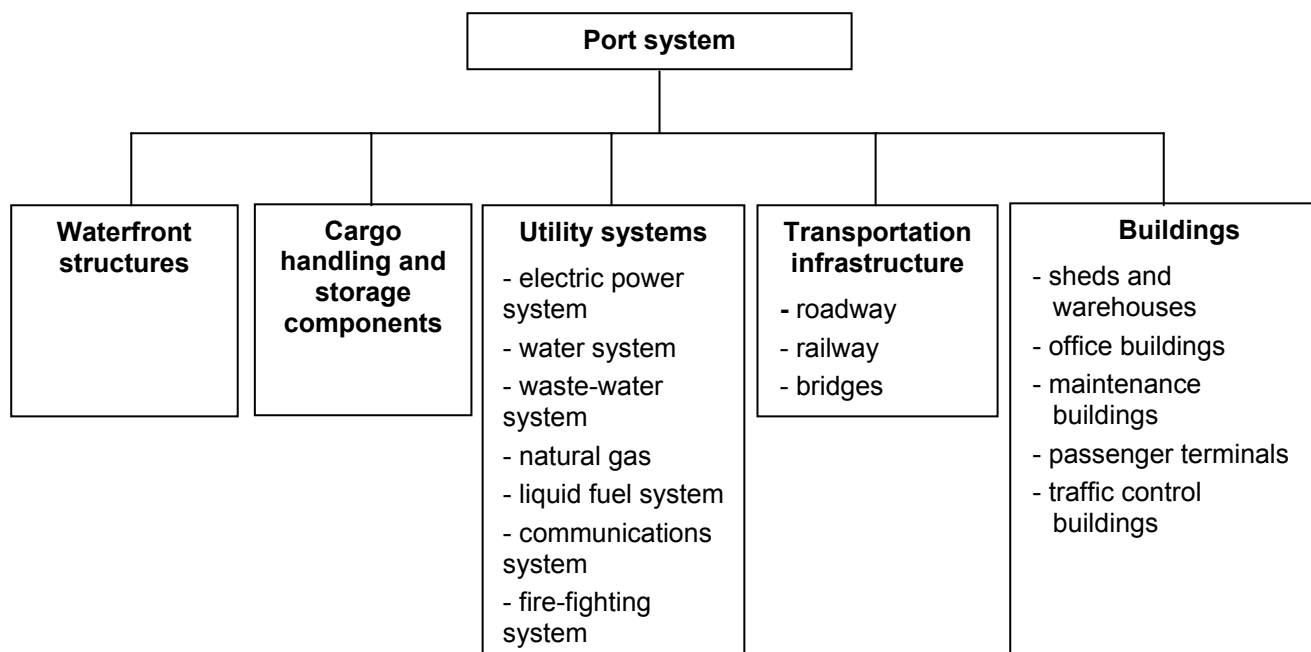


Fig. 3.1 Breakdown of port system components

3.1.1 Waterfront structures

Port waterfront structures are quite broad and varied, and include a range of earthen embankments and berthing structures. The embankments may be homogeneous or multi-lift earth fill, armoured with rock rip rap or other materials, and may possibly be topped with a concrete structure. Berthing structures at ports may be composed of earth retaining walls, pile supported marginal wharves, pile-supported piers, or combinations of these.

Seven specific types of port waterfront structures may be considered:

- Embankments;
- Piles (common element for many types of waterfront and other types of port components);
- Marginal wharves;
- Gravity retaining structures;
- Steel sheet pile wharves;
- Mooring and breasting dolphins;
- Piers.

At least, waterfront structures include (NIBS, 2004):

- Wharves;
- Seawalls;
- Piers.

Typology

From an engineering point of view, waterfront structures are soil-structure systems that consist of various combinations of structural and foundation types. Typological descriptions of port waterfront structures are more or less detailed. Some port structures are “mixed” and can not be fully characterized by a single structural or foundation type.

Wood, steel or concrete piles often support waterfront structures. Many also have batter piles to resist lateral loads from wave action and small impact of vessels. Seawalls are caisson walls retaining earth fill material.

The basic typological parameters are: geometry, section type, construction material, foundation type, existence and type of anchorage. Type of backfill and foundation soil, along with the existence of rubble foundation are primary determinant factors of their seismic behavior (Ichii, 2003).

A more exhaustive typology may be used (Werner, 1998; PIANC, 2001):

Gravity retaining structures along the waterfront (quay walls/piers): concrete block walls (block work), massive walls (monolithic structure), concrete caissons (monolithic structure), cantilever structures (monolithic structure), cellular sheet pile retaining structures, steel plate cylindrical caissons or cribwork quay walls. Founded on rubble and soil or rock.

Sheet pile wharves with auxiliary structures for anchoring or sheet pile with platform (horizontal pile-supported slab). Sheet pile, pile, fill-soil foundation. Often made of steel.

Marginal wharves are pile structures, often partly soil-retaining and with auxiliary structures for anchoring. Deck types are: cast-in-place concrete flat slabs, cast-in-place concrete beam and slab structures, long-span concrete box girder deck system, precast pre-stressed concrete panels, precast normally-reinforced concrete panels, ballasted deck pavement systems. Also, structures on columns with auxiliary structures for horizontal force absorption (sometimes partly soil-retaining).

Piers usually of deck slabs supported on pile caps and piles from wood, steel or concrete (with or without batter piles). Piers (typically perpendicular to the shore line) are distinguished from wharves which parallel a shore line and include an earth or rock dike structure.

Mooring and breasting dolphins: monolithic gravity structures, founded on rubble and soil or rock (sometimes pile foundation), or pile structures (floating dock wharves).

Embankments could be native soils, rock and sand dike with backland fills, bulkheads and sea walls and breakwaters (vertical face and rubble mound types). Breakwaters are further classified into: conventional caisson breakwater with vertical front (monolithic gravity structure, foundation on rubble bedding layer and soil or rock), “vertically composite” caisson breakwater (monolithic gravity structure, foundation on high rubble mound), “horizontally composite” caisson breakwater (monolithic gravity structure with mound of blocks on one side, foundation on bedding layer and soil or rock), “block work” breakwater (block work gravity structure, foundation on bedding layer, or rubble mound, and soil or rock), piled breakwater, multi layer rubble mound breakwater with superstructure (mound with or without monolithic crown wall super structure, mound on soil foundation), reshaped rubble mound (Berm breakwater) (mound without superstructure, mound on soil foundation).

Piles types are: pre-stressed concrete piles, large-diameter pre-stressed concrete cylinder piles, cast-in-drilled-holes (CIDH) concrete piles, steel H-piles, steel pipe piles, large-diameter steel pipe piles, timber piles, vertical pile systems, batter pile systems.

In **Thessaloniki's Port (Greece)**, waterfront structures include concrete gravity quay walls with surface foundation and no anchorage components. The majority is block work, while the new under construction part of Thessaloniki's port includes caisson type structures. Backfill soils and rubble foundation include material aggregates with appropriate grain size distributions.

3.1.2 Cargo handling and storage components

Cranes and cargo handling equipment are large equipment items used to load and unload freight from vessels. Cargo handling and storage components found in ports may be the following:

- Cranes.
- Tanks.
- Other cargo handling and storage components.

Typology

Cranes and cargo handling equipment are described (NIBS, 2004) with respect to whether the cranes are:

- Anchored / Unanchored;
- Stationary / Rail mounted.

They could also be classified according to the cargo capacity and cargo type.

Typological parameters, important when considering interactions between port components, are their power supply type (electric or fuel), foundation type (surface concrete beam or piles) and position (above and/or near waterfront structures or inside the port area).

A more exhaustive typology may be used (Werner, 1998):

- Cranes could be rail, tire and track mounted gantry and revolver cranes, mobile cranes and crane foundations and power supply systems.
- Tanks could be anchored and unanchored, above grade and partially buried, tank foundations and containment berms.
- Other cargo handling and storage components include port equipment (stationary or mounted on rails) and structural systems used for material handling and transport (cranes, conveyors, transfer towers and stacker/reclaimer equipment), tunnels and pipelines, and temporary transitional storage and containment components.

3.1.3 Port infrastructures

Buildings

Buildings at port may be the following:

- Traffic control buildings;
- Passenger terminals;
- Office buildings;
- Maintenance buildings;
- Sheds and warehouses.

They house various functions:

- Administration,
- Security,
- Maintenance,
- Engineering,
- Storage,
- Passenger terminal,
- Other facilities.

Sheds and warehouses

Sheds and warehouses are buildings belonging to the port systems, where goods are stored prior to their use, distribution, or sale. They may be distant from the shoreline, or located on the wharf itself. Sheds and warehouses generally have low human occupancy and contain a variety of cargoes ranging from bulk products to manufactured goods. They are large, open, frame-type structures with long-span roof systems usually braced in 1 or 2 directions. Some have centre columns to reduce roof support spans. Perimeter, division and end walls are usually constructed of tilt-up concrete, masonry or metal siding

See also D3.1 “Fragility functions for common RC building types in Europe” and D3.2 “Fragility functions for masonry buildings in Europe”.

Office buildings

Office buildings at ports consist of single or multi-storey steel, timber, concrete or masonry construction. They are usually extensively occupied and house critical operations associated with computer systems, command centres, and port management, administration, engineering and security functions. After an earthquake, port office buildings may contain facilities where emergency response functions are housed, as well as service support for recovery actions.

See also D3.1 “Fragility functions for common RC building types in Europe” and D3.2 “Fragility functions for masonry buildings in Europe”.

Maintenance buildings

Maintenance buildings at ports are used for repairing equipment or keeping it in proper condition. Maintenance facilities are not usually critical for system functionality as

maintenance activities can be delayed or performed elsewhere. Generally, they are similar to shed and warehouse structures, but with significantly more personnel occupancy, utilities and mechanical equipment. They may also house hazardous chemical and gas storage tanks, fuelling systems, overhead cranes, high roof bays and material storage racks.

See also D3.1 “Fragility functions for common RC building types in Europe” and D3.2 “Fragility functions for masonry buildings in Europe”.

Passenger terminals

Passenger terminals at ports are often long span passenger queuing areas, serving as an important access point to the ships. By nature, human occupancy in passenger terminals could be high. High occupancies require strict code compliance for structure, fire and access/egress. Port passenger terminals may be constructed from concrete, masonry, steel or wood.

See also D3.1 “Fragility functions for common RC building types in Europe” and D3.2 “Fragility functions for masonry buildings in Europe”.

Traffic control buildings

Traffic control buildings are tall, narrow tower-type structures, that are usually steel-frame construction with exterior masonry or other cladding. Heights of these towers can exceed 30m. The upper stories of these towers are usually occupied by observation/ control offices, often with critical operational yard functions or vessel navigation/ communication roles. The towers may also contain important data cabling and radio communication links.

See also D3.1 “Fragility functions for common RC building types in Europe” and D3.2 “Fragility functions for masonry buildings in Europe”.

Older unreinforced masonry and non-ductile concrete frame structures, constructed prior to seismic code changes and improvements or even without seismic design, often exist in port facilities. They do not incorporate modern seismic design and detailing provisions for structural systems with adequate strength, resistance and ductility. They have been used for warehouse, office, maintenance, and passenger terminals buildings.

Utility systems

Electric power system

Electric power is necessary for the operation of cranes, pumps, area lighting, office equipment and buildings, and as an auxiliary supply to operate electrical equipment on board the ships. In addition, emergency back-up power is required in case of commercial power loss.

See also D3.3 “Fragility functions for electric power system elements”.

Water system

A potable water supply is necessary at ports for drinking, food preparation, and sanitation, and to provide an auxiliary potable supply to the ships. Water (which may be non-potable) is also required for cooling equipment.

See also D3.5 “Fragility functions for water and waste-water system elements”.

Waste-water system

Waste disposal is required for sanitary purposes for the port's landslide and ship operations. Adequate drainage of the site is vital to keep the area clear for operations.

See also D3.5 "Fragility functions for water and waste-water system elements".

Natural gas system

Natural gas is used for heating in offices and for food preparation.

See also D3.4 "Fragility functions for gas and oil network elements".

Liquid fuel system

Liquid fuels are used to operate the port's fixed and mobile internal combustion engine-driven equipment and emergency generators, and to fuel ships.

Liquid fuel system consists of fuel storage tanks, buildings, pump equipment, piping, and, sometimes, backup power.

The components may be anchored or un-anchored, with or without backup power.

See also D3.4 "Fragility functions for gas and oil network elements".

Communications system

Telephone and radio communication facilities are essential to the port's normal business operations, communication between landslide operations and ships, and emergency response and recovery.

Fire-fighting system

Fire suppression at wharves, warehouses, buildings and on ships.

See also D3.11 "Fragility functions for fire fighting system elements".

Transportation infrastructures

Roadway system

Roadway access links are commonly required at port facilities to transfer of waterborne cargo and people to and from their landslide origins and destinations.

Roadway components include roadbed embankments, walls, signs, lights, pavements, base and sub-base, bridges and tunnels. Tunnels may be lined or unlined, through alluvium or rock, at any depth below ground surface.

See also D3.7 "Fragility functions for roadway system elements".

Railway system

Railway access links are commonly required at port facilities to transfer of waterborne cargo and people to and from their landslide origins and destinations, although not all ports have rail access.

Railway components include roadbed embankments, walls, ties, rails ballast and switches.

See also D3.8 "Fragility functions for railway system elements".

Bridges

Bridges are a common element of roadway and rail access systems at ports. Offshore port facilities, as well as many inland-waterway piers, wharves and docks, require access bridges that often span across bodies of water.

See also D3.6 “Fragility functions for roadway bridges”.

3.2 GENERAL DESCRIPTION OF EXISTING METHODOLOGIES

Fragility relationships are a critical component of seismic impact assessment. The fragility, or vulnerability, functions relate the severity of shaking to the probability of reaching a level of damage (e.g. light, medium, extensive, near-collapse) to various infrastructure items. The level of shaking can be quantified using numerous shaking parameters, including peak ground acceleration, velocity, displacement, spectral acceleration, spectral velocity or spectral displacement. Each infrastructure item requires a corresponding set of fragilities to determine damage level likelihoods (probability).

In general, fragility functions relate a level of shaking, or system demand, to the conditional probability of a specific system reaching or exceeding a limit state response. A deterministic response, or the vertical line, indicates a lack of uncertainty in the system response. Fragility curves close to vertical indicate a low level of uncertainty, while those with a much higher uncertainty are spread over a much wider range of shaking values.

In the following, existing methodologies for the vulnerability assessment of port facilities are reviewed.

3.2.1 Waterfront structures

The type and degree of seismic damages of the waterfront structures depend upon the typology, the local site conditions, the intensity of the seismic loading and the occurrence of liquefaction. The design factors of safety also play a very important role in their seismic behaviour. In current engineering practice the seismic design of earth retaining structures is usually carried out using empirical methods. According to the quasi totality of seismic codes worldwide, waterfront structures are designed using simplified, pseudo-static or simplified static, force-based equilibrium approaches and pseudo-dynamic techniques (Steedman and Zeng, 1990). An alternative approach developed recently is the use of displacement-based methods. An estimate of earthquake-induced displacement may be obtained by performing simplified dynamic analyses (sliding block method; Newmark, 1965; Richards and Elms, 1979) or alternatively advanced non-linear time-history analyses using numerical finite difference or finite element simulations (full dynamic analysis; Whitman, 1990; Alampalli and Elgamel, 1990; Pitilakis and Moutsakis, 1989; Finn et al., 1992; lai and Kameoka, 1993; Al-Homoud and Whitman, 1999; Green and Ebeling, 2002; Psarropoulos et al., 2005). For static conditions, the prediction of actual earth pressures and permanent displacements, which are necessary for the construction of fragility curves, constitutes a complicated soil-structure interaction problem. In the dynamic response the situation is even more complicated. The dynamic response of the simplest type of retaining wall is depending on the mass and stiffness of the wall, the backfill and the underlying ground, as well as the interaction among these components and the characteristics of the seismic input.

Empirical fragility curves describing earthquake induced damage in waterfront structures are proposed in HAZUS (NIBS, 2004). They describe log-normal cumulative distributions which give the probability of reaching or exceeding certain damage states for a given level of permanent ground displacement (PGD). Five damage states are used (none-ds₁, slight/minor-ds₂, moderate-ds₃, extensive-ds₄ and complete-ds₅). The damage index is based on the description of the type and extent (level) of structural damage and serviceability state. In this case, no distinction between the different wall typologies and no specification of the type and source of ground displacement (deformation due to ground shaking or ground failure) are made.

Analytical methods have also been used for the vulnerability assessment of quay walls (Roth and Dawson, 2003; Roth et al., 2003). The standard “structural-engineering approach” for retaining structures seismic design, relies on soil-structure interaction models; alternatively, a full dynamic analysis can be performed (Pathmanathan et al., 2007; Pasquali et al., 2008; Li Destri Nicosia, 2008; Green et al., 2008). This kind of analysis provides a useful insight of the seismic behavior of waterfront structures but cannot be easily applicable for a straightforward vulnerability assessment of different wall typologies and foundation conditions, under different levels of seismic excitation.

Ichii 2003 and 2004 proposed several analytical fragility curves for the assessment of direct earthquake-induced damage to gravity-type quay walls using simplified dynamic finite element analysis, considering also the occurrence of liquefaction phenomena. Different vulnerability curves are given in the form of log-normal probability distributions for different peak ground acceleration levels (PGA). Seismic performance of quay walls appears to be governed by the following parameters (Fig. 3.2):

- Width-to-height ratio of the wall (W/H).
- Normalized thickness of sand deposit ($D1/H$). Intermediate deposits to the stiff subsoil layers can be either natural or artificial.
- Equivalent N_{SPT} values (N_{65}) of sand deposits below and behind the wall (corrected values for the effective vertical stress of 65 kPa in terms of an equivalent relative density).

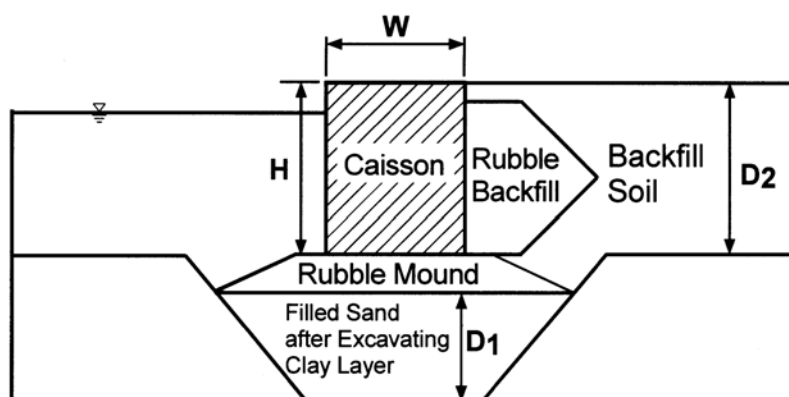


Fig. 3.2 Definition of fragility curves' parameters for quaywalls (Ichii, 2003)

Five damage states are used (none, degree I, degree II, degree III and degree IV). The damage index used is the normalized seaward displacement and restoration cost. The proposed vulnerability curves are expressed in the form of two parameter (median and log-

standard deviation) lognormal distribution functions $\{F(a)=\Phi[\ln(a/c)/\zeta]\}$. Fragility curve parameters for gravity type quay walls according to Ichii (2003) are provided in Table 3.1, while Fig. 3.3 illustrates a set of example fragility curves ($W/H=0,9$, $D_1/H=0,5$, $N_{65}=10$).

Table 3.1 Parameters of fragility curves for gravity-type quaywalls (Ichii, 2003)

Equivalent SPT N values	Aspect ratio (W/H)	Normalized thickness of sand deposit (D_1/H)	Degree I		Degree II		Degree III		Degree IV	
			c (gal)	ζ	c (gal)	ζ	c (gal)	ζ	c (gal)	ζ
5	0.9	0	160.1	1.12	414.8	0.5	615.6	0.38	689.7	0.25
8	0.9	0	246.3	0.65	438.5	0.4	611.9	0.33	663.7	0.19
10	0.9	0	291.6	0.5	453.7	0.36	607.9	0.28	649.2	0.17
15	0.9	0	337.5	0.45	505.2	0.25	608	0.16	635.3	0.09
20	0.9	0	388.2	0.37	545.7	0.18	619.7	0.12	678.6	0.11
25	0.9	0	412.7	0.34	574.4	0.15	631.9	0.09	2650.1	0.29
5	0.9	1	0.1	7.05	0.1	8.27	0.1	9.39	0.2	11.68
8	0.9	1	11.3	3.27	146.3	1.17	276.9	0.79	366.7	0.65
10	0.9	1	93.6	1.4	268.1	0.65	390.1	0.46	462.6	0.39
15	0.9	1	209.6	0.75	392.5	0.42	511	0.29	589.9	0.22
20	0.9	1	353.1	0.41	506.6	0.23	600.5	0.16	617.7	0.08
25	0.9	1	404.9	0.33	560.5	0.19	617.7	0.1	1751.9	0.49
15	0.65	0	262.7	0.55	429.2	0.35	555.1	0.28	625.8	0.21
15	0.9	0	337.5	0.45	505.2	0.25	608	0.16	625.3	0.09
15	1.05	0	375.4	0.38	547.2	0.22	629.6	0.14	713.9	0.12
15	0.65	1	208.1	0.74	378.8	0.41	484.4	0.31	568.8	0.26
15	0.9	1	209.6	0.75	392.5	0.42	511	0.29	589.9	0.22
15	1.05	1	215.5	0.73	400	0.41	512.5	0.29	587.5	0.2
10	0.9	0.5	145.8	1.01	307.9	0.53	414.8	0.45	499.8	0.41
20	0.9	0.5	375.2	0.37	523.2	0.19	609.8	0.14	638.7	0.09

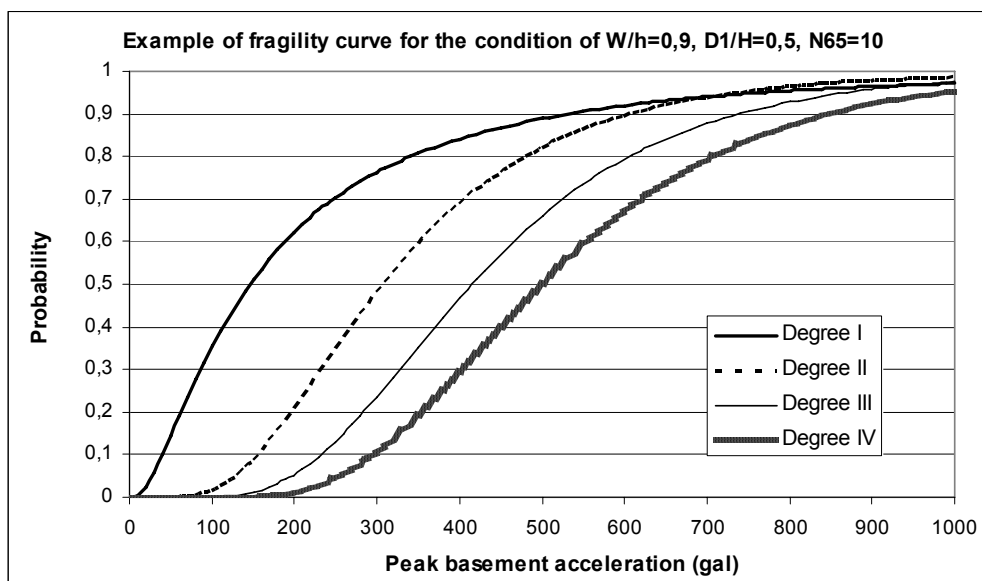


Fig. 3.3 Example fragility curves ($W/H=0.9$, $D_1/H=0.5$, $N_{65}=10$) (Ichii, 2003)

Kakderi and Pitilakis (2010) proposed fragility curves for waterfront/ retaining structures for ground shaking without the presence of liquefaction, using available data from past earthquakes' damages in Europe and worldwide and numerical analysis of typical cases. Typical waterfront structures, with different geometry, foundation soil conditions and seismic excitations, are studied using appropriate numerical modeling (2D finite element analysis). The corresponding damage levels are estimated with respect to the induced residual displacements and the seismic response of the soil-structure system. Considering aleatory uncertainties of the parameters involved, analytical fragility curves were constructed for the different types of waterfront structures and foundation conditions. The computed analytical fragility curves were finally compared with the validated empirical ones, in order to propose fragility functions and corresponding damage levels for gravity waterfront/ retaining structures based on European distinctive features. The proposed fragility curves are given in terms of two parameters (median and standard deviation β) log-normal cumulative distributions for different levels of peak ground acceleration levels (PGA) at rock outcrop conditions. Four damage states are used (none, minor, moderate and extensive) in respect to the normalized residual horizontal seaward displacement. Fragility curves are applicable to gravity-type (monolithic) quay walls, only for ground shaking, without the presence of liquefaction. In total 4 different classes are considered based on:

- Wall height H ($>$ and ≤ 10 m).
- Soil foundation conditions (V_s values) (soil types B and C according to EC8).

Na et al. (2008) investigate the effect of the liquefaction and lateral spread on the seismic response of (gravity type) caisson quay walls, using nonlinear dynamic analyses of soil-structure system (effective stress analysis). In this study, a 2D numerical model, representing PC1 berth located in Port Island, Kobe and damaged in the 1995 Kobe earthquake, has been developed and used to simulate seismic behaviour. The propagation of uncertainties of soil-structure system to the quay wall and uncertainty of ground motions have also been investigated. To identify and rank the significant sources of basic uncertainties, residual horizontal displacement (RHD) of top sea side corner of a quay wall is introduced as a seismic demand of a port structure. Two sets of fragility curves representing

an original and a retrofitted structure are presented to assess the response of the specific berth typologies for different levels of peak ground acceleration (PGA) at stiff soil to rock site. Five damage states are used (none, damage state I-serviceable, damage state II-repairable, damage state III-near collapse and damage state IV-collapse) in respect to the normalized residual horizontal seaward displacement. However, the parameters of the fragility relations are not provided. Na and Shinozuka (2009) are further presenting a methodology to estimate the effects of the earthquake on the performance of the operation system of a container terminal in seaports by integrating simulation models for terminal operation and fragility curves of port components in the context of seismic risk analysis. Finally, system fragility for a container curves are developed terminal based on fragility curves of independent wharf components.

Focusing on pile-supported wharves Na et al. (2009b) evaluate seismic behaviour of port structures recognizing that most of the parameters controlling the properties of soil are of a random nature. The response of such structures inherently presents a complex soil-structure interaction (SSI) problem involving ground shaking, pile-failure mechanism, and liquefaction and lateral spreading in backfill and sand layers. In this study, using a representative model of a typical pile-supported wharf in the west coast of United States and considering an ensemble of ground motions with different hazard levels, effect of soil parameter uncertainty on seismic response is evaluated. For numerical simulations, an effective stress analysis method has been utilized. In order to investigate the effect of soil parameter uncertainty on seismic response, random samples are generated and nonlinear time history analysis is carried out repeatedly for each realized sample. Finally, the effect of parameter uncertainty is demonstrated in terms of seismic fragility curves. Two sets of fragility curves are proposed for different levels of peak ground acceleration (PGA); with (probabilistic) or without (deterministic) consideration of uncertainty in soil parameters. Five damage states are used (none, damage state I-serviceable, damage state II-repairable, damage state III-near collapse, and damage state IV-collapse) defined in terms of the residual horizontal displacements of a dike and a deck. However, parameters of the fragility relations are not provided and the results of this study can be only utilized to evaluate the seismic vulnerability of similar pile supported wharves.

In the study performed by Ko et al. (2010), the seismic fragility analysis for the sheet pile wharves of the Hualien Harbor in Taiwan was performed using 2D finite element nonlinear dynamic analysis. The time histories of several representative earthquake events that actually occurred in Taiwan, including the 1999 Chi-Chi Earthquake, were used. The obtained seismic responses of the sheet pile wharves when subjected to earthquakes of different intensities were used for the deduction of the fragility curves. Moreover, these fragility curves were parameterized assuming they can be well approximated by two parameters (median and standard deviation β) lognormal cumulative probability functions. Four damage states are used (none, minor, moderate and extensive) to describe the seismic response in terms of the maximum residual displacement at the top of the sheet pile wall subjected to different levels of peak ground acceleration (PGA) at free-field conditions. However, the proposed fragility curves refer to the specific sheet pile cross sections of wharves of the Hualien Harbor in Taiwan (anchors at the upper part and soil embedment at the lower part, with or without additional gravity retaining wall, heights 16-20m and Stiff, non-cohesive foundation soil).

Finally, an ongoing research is being carried out under the NEESR Grand Challenge Project, aiming to develop and implement improved procedures for seismic risk management of container port systems. Central to these procedures is a risk analysis framework for

deterministic or probabilistic assessment of economic risks due to earthquake-induced interruption of shipping operations for an entire system of berths that comprises an overall port system (Werner et al., 2009). As part of the adopted probabilistic assessment, fragility models for wharf structures will be proposed. In these models, damage states are represented not only in terms of the extents, types, and locations of damage throughout the structure, but also in terms of the costs and times to repair this damage, and the ability of the wharf to accommodate shipping demands as the repairs proceed over time after the earthquake. Fig. 3.4 illustrates a typical form of this fragility model. An alternative representation of this model can be based on use of joint probability density functions. The wharf fragility modelling process includes development of a three-dimensional nonlinear model of the wharf structure (pile supported wharves). The work is still under development.

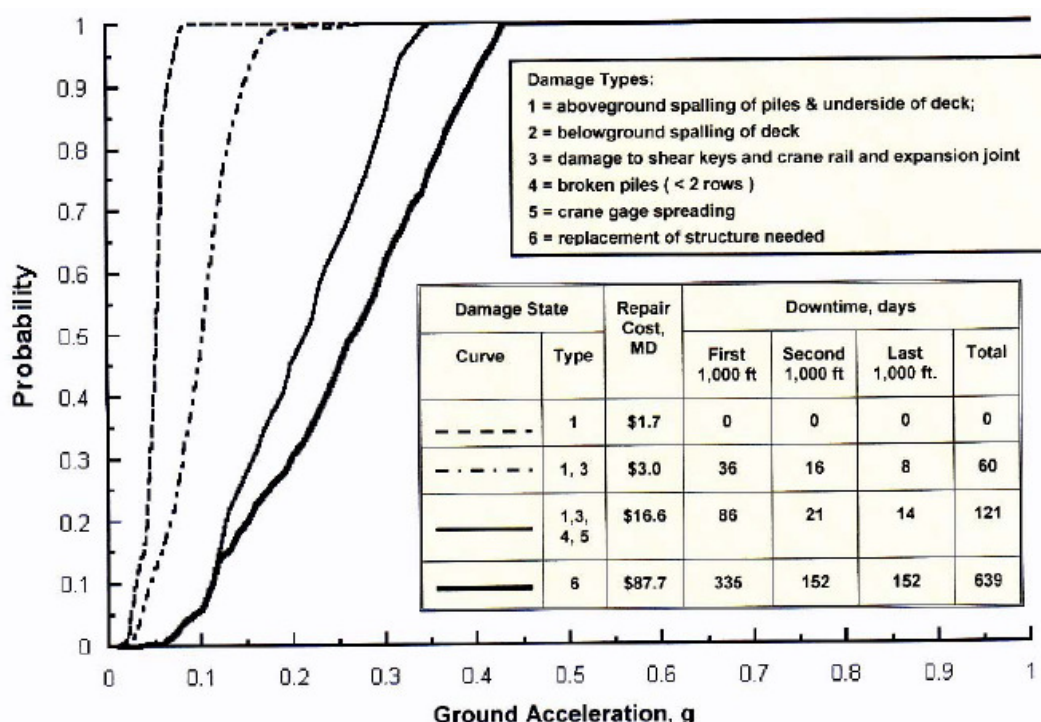


Fig. 3.4 Illustrative wharf fragility model (a wharf structure that is 3,000 ft long, assuming that 1,000 ft of wharf length is required to accommodate one ship) (Werner et al., 2009)

3.2.2 Cargo handling and storage components

The performance of cranes during earthquakes is critical, since crane damage and subsequent downtime has a major impact on indirect losses and post-disaster recovery. Identifying the key parameters which negatively affect the seismic response of cranes is the first step to developing fragility relationships, and ultimately, performance-based design recommendations.

Empirical fragility curves describing earthquake induced damage to cargo handling and storage components are proposed in HAZUS (NIBS, 2004). They describe log-normal cumulative distributions which give the probability of reaching or exceeding certain damage states for a given level of peak ground acceleration (PGA) and permanent ground displacement (PGD). Four damage states are used (none-ds₁, slight/minor-ds₂, moderate-

ds₃ and extensive/complete-ds₄). The damage index is based on the description of the type and extent (level) of structural damage and serviceability state. A distinction is made between stationary (anchored) and rail-mounted (un-anchored) cranes.

3.2.3 Port infrastructures

Buildings

See D3.1 “Fragility functions for common RC building types in Europe” and D3.2 “Fragility functions for masonry buildings in Europe”.

Utility systems

Electric power system

See D3.3 “Fragility functions for electric power system elements”.

Water system

See D3.5 “Fragility functions for water and waste-water system elements”.

Waste-water system

See D3.5 “Fragility functions for water and waste-water system elements”.

Natural gas system

See D3.4 “Fragility functions for gas and oil network elements”.

Liquid fuel system

Empirical fragility curves that describe earthquake induced damage to fuel facilities are proposed in HAZUS (NIBS, 2004). Damage functions for fuel facilities are similar to those developed for railway fuel facilities. They describe log-normal cumulative distributions which give the probability of reaching or exceeding certain damage states for a given degree of peak ground acceleration (PGA) and permanent ground deformation (PGD). Five damage states are used (none-ds₁, slight/minor-ds₂, moderate-ds₃, extensive-ds₄ and complete-ds₅). The damage index is based on the description of the type and extent (level) of structural damage and serviceability state. Five types of fuel facilities are considered:

- anchored equipment with backup power
- anchored equipment without backup power
- unanchored equipment with backup power
- unanchored equipment without backup power
- facilities with buried tanks.

In the framework of a recent Greek research project (SRMLIFE, 2003-2007) fragility curves for liquid fuel facilities are derived using the same fault tree analysis as in HAZUS (NIBS, 2004) while modifying the fragility curves of the sub-components. For the building sub-component, the fragility curves proposed by Kappos et al. (2006) for European typologies are used (R/C dual system, regularly infilled, low-rise building). The derived fragility curves describe log-normal cumulative distributions which give the probability of reaching or exceeding certain damage states for a given degree of peak ground acceleration (PGA) and permanent ground deformation (PGD). Five damage states are used (none-ds₁, slight/minor-

ds₂, moderate-ds₃, extensive-ds₄ and complete-ds₅). The damage index is based on the description of the type and extent (level) of structural damage and serviceability state. Seven types of fuel facilities are considered:

- unanchored equipment with back-up power – building with low seismic code design
- unanchored equipment without back-up power– building with low seismic code design
- unanchored equipment with back-up power– building with medium seismic code design
- unanchored equipment without back-up power– building with medium seismic code design
- unanchored equipment with back-up power– building with high seismic code design
- unanchored equipment without back-up power– building with high seismic code design
- facilities with buried tanks.

See also D3.4 “Fragility functions for gas and oil network elements”.

Communications system

The major components of a communication system are:

- Central offices and broadcasting stations (this includes all subcomponents such as central switching equipment).
- Transmission lines (these include all subcomponents such as equipment used to connect central office to end users).
- Cabling (low capacity links).

Central offices and broadcasting stations are the only components of the communication system which are usually considered to be fragile. Other components, such as cables and other lines, usually have enough/ slack to accommodate ground shaking and even moderate amounts of permanent ground deformations.

Empirical fragility curves that describe earthquake induced damage to central offices and broadcasting stations are proposed in HAZUS (NIBS, 2004). They describe log-normal cumulative distributions which give the probability of reaching or exceeding certain damage states for a given degree of peak ground acceleration (PGA). Five damage states are used (none-ds₁, slight/minor-ds₂, moderate-ds₃, extensive-ds₄ and complete-ds₅). The damage index is based on the description of the type and extent (level) of structural damage and serviceability state. Two types of communication facilities are considered:

- facilities with anchored components
- facilities with unanchored components.

In the framework of a recent Greek research project (SRMLIFE, 2003-2007) fragility curves for communication facilities are derived using the same fault tree analysis as in HAZUS (NIBS, 2004) while modifying the fragility curves of the sub-components. For the building sub-component, the fragility curves proposed by Kappos et al. (2006) for European

typologies are used (R/C dual system, low and middle rise building), while all the sub-components equipment is supposed to be anchored. The derived fragility curves describe log-normal cumulative distributions which give the probability of reaching or exceeding certain damage states for a given degree of peak ground acceleration (PGA). Five damage states are used (none-ds₁, slight/minor-ds₂, moderate-ds₃, extensive-ds₄ and complete-ds₅). The damage index is based on the description of the type and extent (level) of structural damage, restoration cost (as a percentage of the replacement cost) and serviceability state. Four types of fuel facilities are considered:

- anchored components - low-rise building with low seismic code design
- anchored components - mid-rise building with low seismic code design
- anchored components - low-rise building with high seismic code design
- anchored components - mid-rise building with high seismic code design

Fire-fighting system

See D3.11 “Fragility functions for fire fighting system elements”.

Transportation infrastructures

Roadway system

See D3.7 “Fragility functions for roadway system elements”.

Railway system

See D3.8 “Fragility functions for railway system elements”.

Bridges

See D3.6 “Fragility functions for roadway bridges”.

3.3 DAMAGE STATES

3.3.1 Waterfront structures

Quay walls

Parameters defining damage states of quay walls are:

- Level of structural damage (HAZUS, NIBS, 2004).
- Serviceability (HAZUS, NIBS, 2004; OCDI, 2002).
- Normalized seaward displacement / sliding (Ichii, 2003; PIANC, 2001).
- Residual tilting towards the sea (PIANC, 2001).
- Level of apron damage (differential settlement, residual tilting) (PIANC, 2001).
- Permanent / residual displacement at top of wall (Uwabe, 1983).
- Horizontal displacement of quay wall (OCDI, 2002).

Especially for caisson-type quay walls:

- Residual displacements of the caisson and apron (PIANC, 2001).
- Peak response stresses / strains of cell and cell joint (PIANC, 2001).

Sheet-pile quay walls

Parameters defining damage states of sheet-pile walls are:

- Normalized seaward displacement / sliding of sheet-pile wall and apron (PIANC, 2001).
- Peak response stresses / strains of sheet-pile wall and tie-rod (PIANC, 2001).
- (Maximum) residual displacement at top of sheet pile (Uwabe, 1983; Gazetas et al., 1990).
- Horizontal displacement (OCDI, 2002).
- Serviceability (OCDI, 2002).

Pile-supported wharves

Parameters defining damage states of pile-supported wharves are:

- Residual displacements (differential settlement between deck and land behind, residual tilting towards the sea) (PIANC, 2001).
- Peak response of piles (PIANC, 2001).

It is noticed that according to EC8 – Part 5, possible failure modes of earth retaining structures are considered to be bending for flexible structures and sliding and/or rotation for gravity structures. The design of earth retaining structures should ensure the following requirements:

- Stability of foundation soil (overall stability, local soil failure by sliding and/or bearing capacity failure).
- Resistance of anchorage.
- Structural strength.

3.3.2 Cargo handling and storage components

Damage states for cargo handling and storage components are based on the description of the type and extent (level) of structural damage and serviceability state.

3.4 INTENSITY INDEXES

The main issue that arises is the question of what ground motion intensity parameter best captures the response of each element and minimizes the dispersion of that response. This is also related to the approach that is followed for the derivation of fragility curves. For example as the empirical ones relate the observed damages with the seismic intensity, the latter is better described based on records of seismic motion, and thus PGA or PGV are more suitable with lower uncertainties.

There is a wide range of intensity descriptors used to assess vulnerability (and losses) through the development of adequate fragility curves. The following is a comprehensive list of the different descriptors used for the components in harbor systems.

Table 3.2 Intensity measures for the vulnerability assessment of harbor elements

Element at risk	Reference	Intensity measure	Comments
Waterfront structures	NIBS, 2004	PGD*	Empirical fragility curves for waterfront structures. No distinction between the different typologies. No specification of the type and source of ground displacement (deformation due to ground shaking or ground failure).
	Ichii, 2003, 2004	PGA	Analytical fragility curves for gravity type quay walls. Simplified dynamic finite element analysis, considering also the occurrence of liquefaction phenomena.
	Kakderi and Pitolakis, 2010	PGA	Analytical fragility curves for ordinary gravity quay walls/ retaining structures' typologies commonly used in Europe, exclusively for ground shaking (no liquefaction and ground failure).
	Ko et al., 2010	PGA	Analytical fragility curves for sheet pile wharves of the Hualien Harbor in Taiwan.
	Na et al., 2009b	PGA	Analytical fragility curves for pile-supported wharves. Liquefaction and lateral spreading in backfill and sand layers is taken into consideration.
	Na and Shinozuka, 2009; Na et al., 2008	PGA	Analytical fragility curves for gravity-type (caisson) quay wall, considering also the occurrence of liquefaction phenomena (effective stress analysis).
Cranes and cargo handling equipment	NIBS, 2004	PGA, PGD*	Expert judgment

PGD*: Permanent Ground Displacements

3.5 PERFORMANCE INDICATORS

The possible consequences of damaging earthquakes (Table 3.3) may be reduction or disruption of transport capacity, requiring some repairs. It is also possible to describe impacts on system dysfunction (Werner, 1995).

Table 3.3 Possible consequences of earthquake on port transportation system.

Serviceability		Seismic performance (Werner, 1995)	
No transportation available	Not repairable	Uncontrolled Collapse control	Beyond collapse control. Significant damage may occur that may not be repairable but will not be sufficient to endanger the life safety of occupants or users of the component.
Reduced transportation	Operational after repairs	Damage control	Requires that damage may occur but should be repairable, controllable and within acceptable limits.
Nominal transportation	Operational without repairs	Collapse control	No loss of function and only minor or negligible damage.

For the individual components, the following indicators could be used:

3.5.1 Quay wall

- ratio of length functioning
- available docking length per ship category
- residual displacement and tilting
- differential settlement on apron and between apron and non-apron areas

3.5.2 Cranes - Cargo handling equipment

- differential displacement of parallel track (derailment, misalignment or toppling)
- peak response of structural members and equipment

4 Fragility functions for harbor elements

4.1 STATE-OF-THE-ART FRAGILITY CURVES PER COMPONENT

A review of existing fragility functions for port systems' components is presented (i.e. waterfront structures, cargo handling and storage components, and port infrastructures). For port infrastructures (building structures, utility and transportation systems), there are no specific fragility studies (except for fuel facilities), and their vulnerability assessment is performed using available fragility functions for lifeline systems' components. Tables 4.1, 4.2 and 4.3 present the review of existing fragility functions for quay walls, cargo handling and storage components and fuel facilities respectively.

Table 4.1 Review of existing fragility functions for quay walls

Reference	Methodology	Classification	Earthquake descriptor	Damage States and Index
NIBS, 2004	HAZUS – empirical fragility functions. Two parameters (median and standard deviation β) log-normal cumulative distributions.	No distinction between the different wall typologies.	Permanent Ground Deformation (PGD) (ground failure)	Five damage states: None (ds_1), slight/minor (ds_2), moderate (ds_3), extensive (ds_4) and complete (ds_5). Index: Description of the type and extent (level) of structural damage and serviceability state.
Ichii, 2003, 2004	Analytical fragility curves for gravity-type (caisson) quay walls using simplified dynamic finite element analysis, considering also the occurrence of liquefaction phenomena. Two parameters (median and standard deviation β) log-normal cumulative distributions.	Gravity-type (caisson) quay walls. In total 20 different classes based on: - Equivalent NSPT value below and behind the wall (range: 5-25) - Aspect ratio of the wall (W/H) (range:0.65-1.05) - Normalized depth of the sand deposit below the wall (D1/H) (range: 0-1)	Peak Basement Acceleration (rock outcrop conditions)	Five damage states: None, degree I, degree II, degree III and degree IV. Index: Normalized seaward displacement and restoration cost.
Kakderi and Pitilakis, 2010	Analytical fragility curves for gravity-type (monolithic) quay walls using 2D finite element analysis, only for ground shaking, without the presence of liquefaction. Two parameters (median and standard deviation β) log-normal cumulative distributions.	Gravity-type (monolithic) quay walls. In total 4 different classes based on: - Wall height H (> and \leq 10m). - Soil foundation conditions (V_s values) (soil types B and C according to EC8).	Peak Ground Acceleration (rock outcrop conditions)	Four damage states: None, minor, moderate and extensive. Index: Normalized residual horizontal seaward displacement.

Reference	Methodology	Classification	Earthquake descriptor	Damage States and Index
Ko et al., 2010	Analytical fragility curves for sheet pile wharves of the Hualien Harbor in Taiwan using 2D finite element nonlinear dynamic analysis. Two parameters (median and standard deviation β) log-normal cumulative distributions.	Two sheet pile cross sections of wharves of the Hualien Harbor in Taiwan (anchors at the upper part and soil embedment at the lower part, with or without additional gravity retaining wall). - H = 16 and 20 m. - Stiff, non-cohesive foundation soil.	Peak Ground Acceleration (PGA) at free-field conditions	Four damage states: None, minor, moderate and extensive. Index: Maximum residual displacement at the top of the sheet pile wall.
Na et al., 2009b*	Analytical approach (effective stress analysis method, nonlinear time history analysis). Liquefaction and lateral spreading in backfill and sand layers is taken into consideration.	- Pile-supported wharves. Two sets of fragility curves: with (probabilistic) or without (deterministic) consideration of uncertainty in soil parameters.	Peak Ground Acceleration (PGA)	Five damage states: None, Damage State I (Serviceable), Damage State II (Repairable), Damage State III (Near Collapse), Damage State IV (Collapse). Index: Residual horizontal displacements of a dike and a deck.
Na and Shinozuka, 2009; Na et al., 2008**	Analytical approach using numerical model (Nonlinear time history analysis), considering also the occurrence of liquefaction phenomena (effective stress analysis).	- Gravity-type (caisson) quay wall Two sets of fragility curves representing an original and a retrofitted structure	Peak Ground Acceleration (PGA) (stiff soil to rock site)	Five damage states: None, Damage State I (Serviceable), Damage State II (Repairable), Damage State III (Near Collapse), Damage State IV (Collapse). Index: Normalized residual horizontal seaward displacement.

* Parameters of the fragility relations are not provided.

** Parameters of the fragility relations are not provided. A methodology is presented to develop system fragility curves for a container terminal based on fragility curves of independent wharf components.

Table 4.2 Review of existing fragility functions for cargo handling and storage components

Reference	Methodology	Classification	Earthquake descriptor	Damage States and Index
NIBS, 2004	HAZUS – empirical fragility functions. Two parameters (median and standard deviation β) log-normal cumulative distributions.	A distinction is made between stationary (anchored) and rail-mounted (un-anchored) cranes.	Peak Ground Acceleration (PGA) and Permanent Ground Deformation (PGD).	Four damage states: None (ds_1), slight/minor (ds_2), moderate (ds_3) and extensive/complete (ds_4). Index: Description of the type and extent (level) of structural damage and serviceability state.

Table 4.3 Review of existing fragility functions for fuel facilities

Reference	Methodology	Classification	Earthquake descriptor	Damage States and Index
NIBS, 2004	HAZUS – empirical fragility functions. Two parameters (median and standard deviation β) log-normal cumulative distributions.	Five types of fuel facilities: *** 1) anchored equipment and backup power 2) anchored equipment but no backup power 3) unanchored equipment and backup power 4) unanchored equipment and no backup power 5) facilities with buried tanks.	Peak Ground Acceleration (PGA) and Permanent Ground Deformation (PGD)	Five damage states: None (ds_1), slight/minor (ds_2), moderate (ds_3), extensive (ds_4) and complete (ds_5). Index: Description of the type and extent (level) of structural damage.

D3.9 - Fragility functions for harbor elements

Reference	Methodology	Classification	Earthquake descriptor	Damage States and Index
SRM-LIFE, 2003-2007	Empirical fragility functions. Two parameters (median and standard deviation β) log-normal cumulative distributions.	<p>Seven types of fuel facilities: ***</p> <ol style="list-style-type: none"> 1) unanchored equipment with back-up power – building with low seismic code design 2) unanchored equipment without back-up power– building with low seismic code design 3) unanchored equipment with back-up power– building with medium seismic code design 4) unanchored equipment without back-up power– building with medium seismic code design 5) unanchored equipment with back-up power– building with high seismic code design 6) unanchored equipment without back-up power– building with high seismic code design 7) facilities with buried tanks. 	Peak Ground Acceleration (PGA) and Permanent Ground Deformation (PGD)	<p>Five damage states: None (ds_1), slight/minor (ds_2), moderate (ds_3), extensive (ds_4) and complete (ds_5).</p> <p>Index: Description of the type and extent (level) of structural damage.</p>

*** Anchored equipment in general refers to equipment designed with special seismic tiedowns or tiebacks, while unanchored equipment refers to equipment designed with no special considerations other than the manufacturer's normal requirements.

4.2 VALIDATION / ADAPTATION / IMPROVEMENT

4.2.1 Validation of vulnerability models for quay walls

The evaluation and examination of the reliability of existing fragility curves has been performed based on the actual seismic performance of the quaywalls of the city of Lefkas, which sustained significant deformations during the 2003 earthquake. The newly constructed quay walls in the Marina suffered minor to moderate damages with observed relative residual seaward displacements of the order of 12 cm to 15cm. There is some evidence that at least in one location a partial liquefaction of the foundation subsoil occurred (Margaris et al., 2003). This study was primarily aiming at developing fragility curves for small quaywalls based on Greek data (typology and construction practice) (Kakderi et al., 2006). One of the main input parameters were the results of a site-specific ground motion analysis performed for the old city of Lefkas and the Marina area and for the specific scenario earthquake. The latter aimed at the estimation of site effects necessary for the vulnerability assessment of spatially distributed lifeline systems.

The geotechnical classification of soil deposits in the area provided a basis for performing of a large number of 1D equivalent linear analyses (32 simplified profiles distributed along a 150mx150m grid). Their validation was performed using aftershock records in a sufficient number of sites within the examined area. Estimated values of peak ground acceleration (PGA) for the equivalent linear analysis vary between 0.64 and 0.41g. The PGA of the available record of the main shock in the city Hospital equals to 0.42g.

In order to account for the effect of liquefaction phenomena on ground motion characteristics, several elastoplastic analyses were performed with the same input motions using the 1D-Cyclic program (Elgamal et al., 2001). Effective stress based analyses give reduced estimates of peak values compared to the results of the equivalent linear analyses (60% reduction for PGA values and 30% for values of PGV). The previous deviations were attributed to the energy dissipation of the liquefied soils along with the different simulation techniques. Along the coastal zone of the city and the marina area, partial or full liquefaction was observed mostly on the surface and quite susceptible soil layers.

Furthermore, the assessment of liquefaction induced permanent ground displacements was performed based on the results of equivalent linear analyses. Simplified empirical methods were used for this purpose (Seed et al., 2003; Youd et al., 2001; Ishihara and Yoshimine, 1992; Bardet et al., 2002). Estimated values and spatial distribution of permanent ground displacements (0-35cm of settlements and 0-25.5cm for lateral spreading) are compatible with the observed ones during the 2003 earthquake, as well as with their spatial distribution.

For the vulnerability assessment of the Lefkas quay walls, the conservative results obtained from the equivalent linear analyses, in terms of both peak ground acceleration and permanent ground displacements, were used as input. Quay walls located in the port and the Marina area are gravity type, concrete retaining structures. The typical cross-section of the Marina's quay walls is illustrated in Fig. 4.1 (newly constructed facility – 2002).

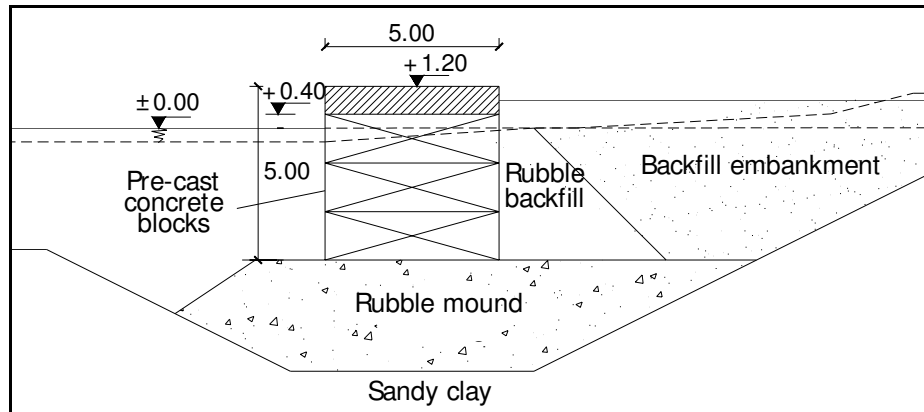


Fig. 4.1 Typical cross-section of Marina-Lefkas quay walls

Fig. 4.2 illustrates the prevailing damage states for Lefkas' quay walls (city's port and marina) using the vulnerability curves proposed from HAZUS. Estimated seismic performance of quay walls (minor to moderate damage) is quite close with the observed one during the 2003 earthquake.

An application of the vulnerability functions proposed from Ichii (2003), was performed for the Marina waterfront structures. The results in terms of estimated damage levels are illustrated in Fig. 4.3. The aforementioned relationships seem to overestimate the quay walls' seismic damage in comparison with their observed behavior.

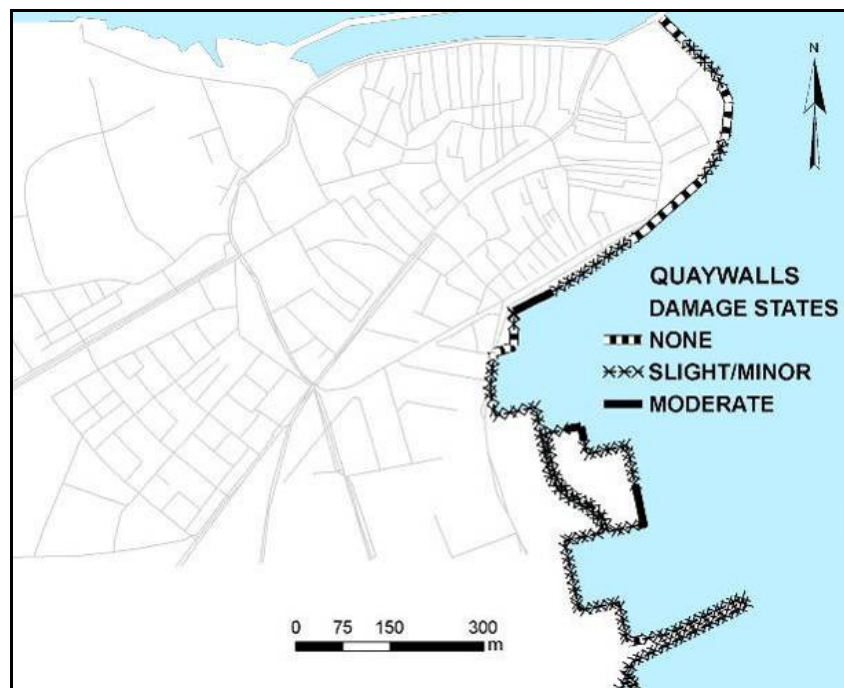


Fig. 4.2 Prevailing damage state for Lefkas quaywalls (HAZUS, NIBS, 2004)

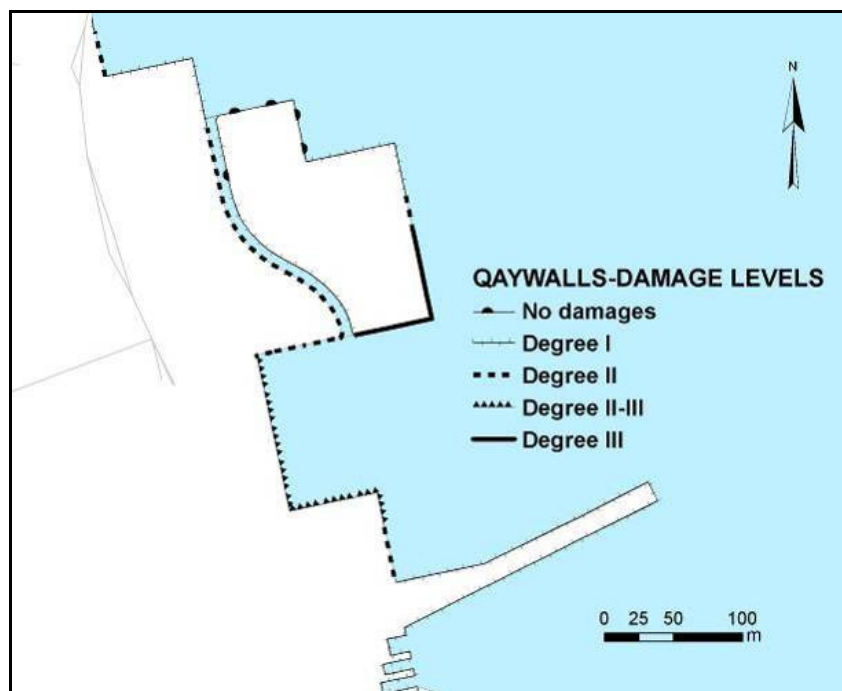


Fig. 4.3 Estimated damage levels for Marina quaywalls (Ichii, 2003)

In conclusion, the following observations can be made:

- The vulnerability assessment and damage state distribution using the HAZUS (NIBS, 2004) relationships is rather compatible with the observed damage.
- Damages based on the vulnerability curves proposed from Ichii (2003) seem to be slightly overestimated.
- The application of Ichii (2003) vulnerability functions requires the knowledge of geotechnical and construction data. Their application is possible in case of lack of specific studies for the estimation of permanent ground displacements.

Finally, it is pointed out that the previous validation studies refer only to the case of induced damages due to ground failure. When considering only the case of ground shaking without the occurrence of liquefaction phenomena, specific fragility functions need to be developed and implemented.

4.3 FINAL PROPOSAL

4.3.1 Waterfront structures

The HAZUS (NIBS, 2004) fragility curves are proposed for the vulnerability assessment of quay walls for the case of ground failure. The description of damage states for waterfront structures is provided in Table 4.4, while the corresponding vulnerability curves are given in Table 4.5 and Fig. 4.4.

Table 4.4 Description of damage states for waterfront structures subject to ground failure

Damage state	Description	Serviceability	
Minor	Minor ground settlement resulting in few piles (for piers/seawalls) getting broken and damaged. Cracks are formed on the surface of the wharf. Repair may be needed.	Reduced use	Operational without repair
Moderate	Considerable ground settlement with several piles (for piers/seawalls) getting broken and damaged.		Operational after repairs
Extensive	Failure of many piles, extensive sliding of piers, and significant ground settlement causing extensive cracking of pavements.	Not usable	
Complete	Failure of most piles due to significant ground settlement. Extensive damage is widespread at the port facility.		Not repairable

Table 4.5 Parameters of fragility curves for waterfront structures subject to ground failure

Damage state	Permanent Ground Displacement (PGD)	
	Median (m)	β (log-standard deviation)
Minor	0.13	0.50
Moderate	0.30	0.50
Extensive	0.43	0.50
Complete	1.09	0.50

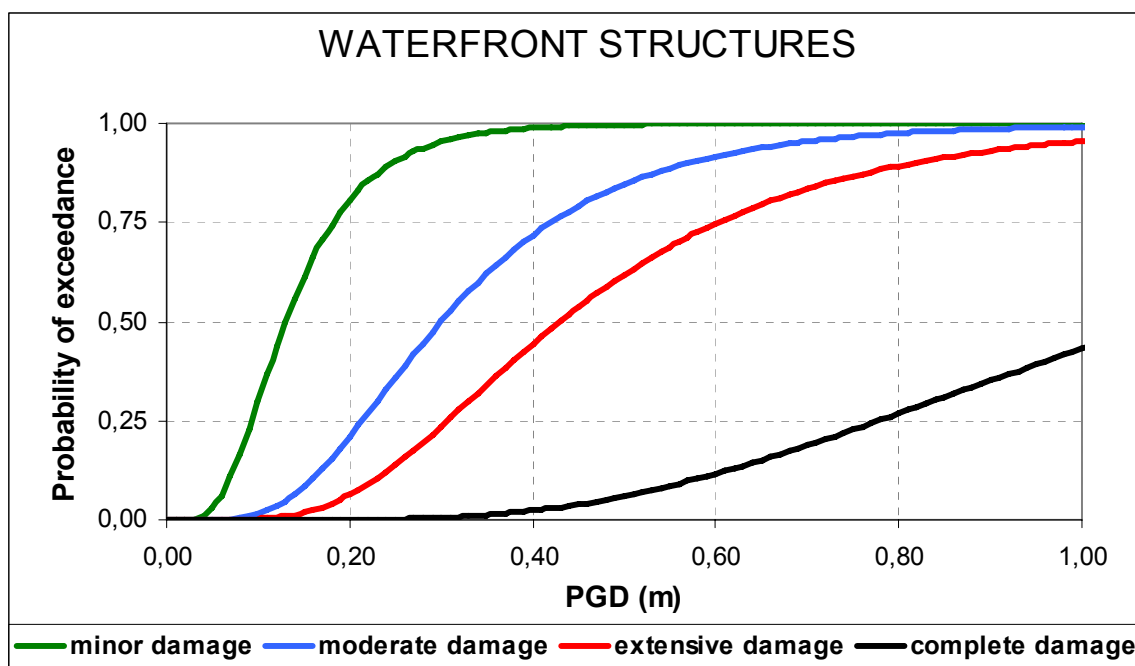


Fig. 4.4 Fragility curves for waterfront structures subject to ground failure

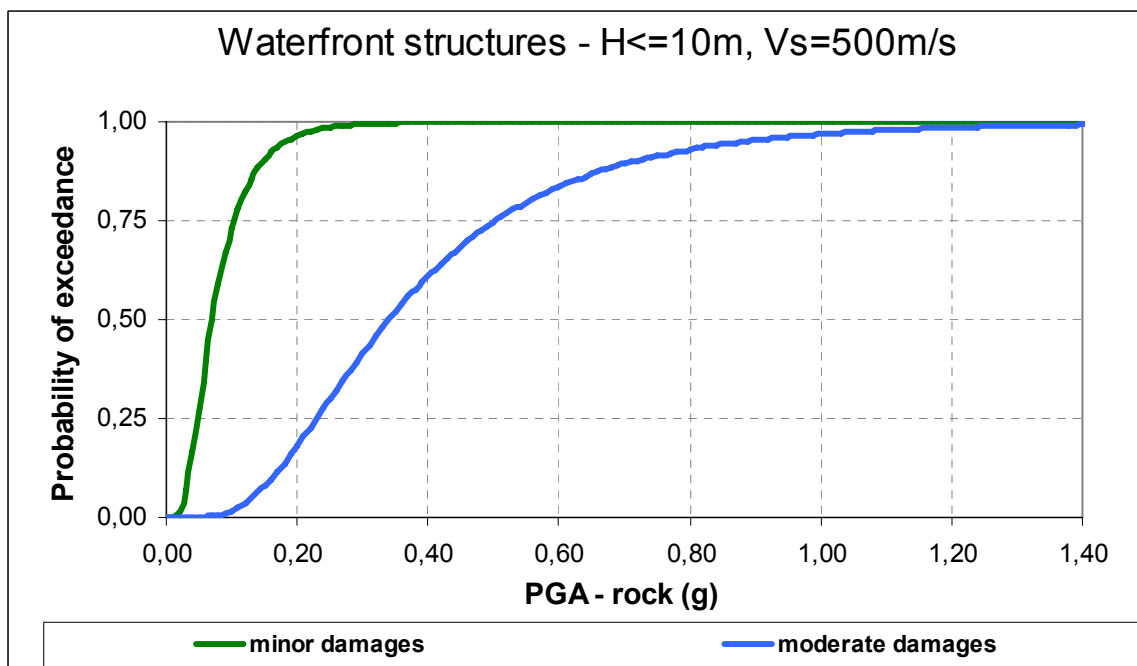
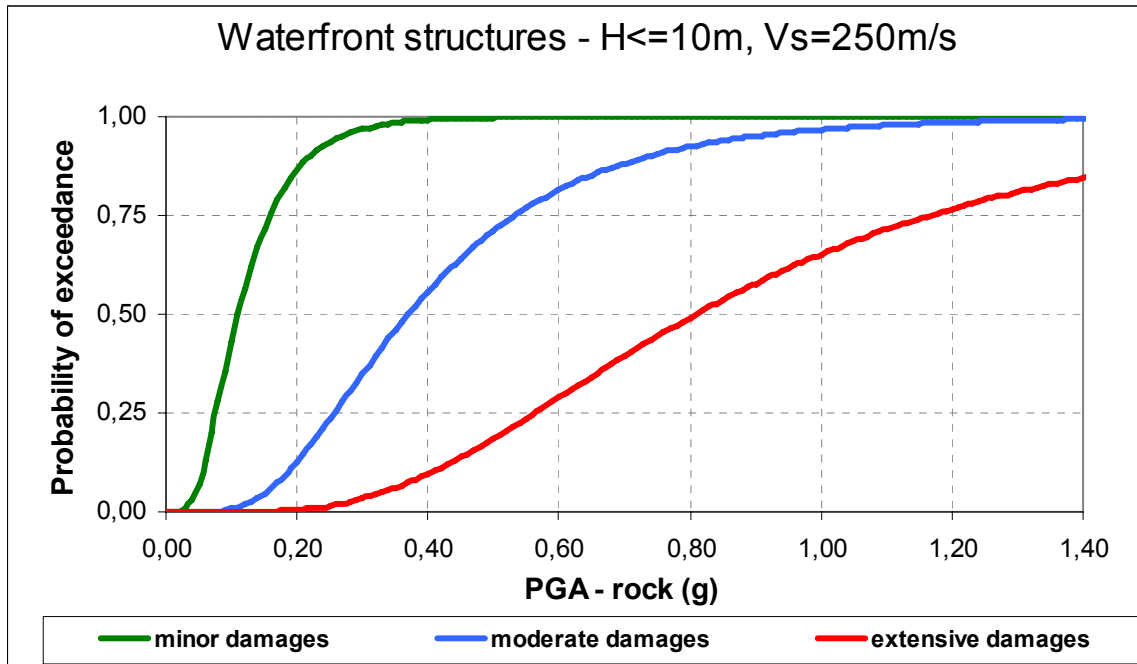
For the case of ground shaking, without considering the occurrence of liquefaction phenomena, the analytically derived fragility curves of Kakderi and Pitilakis (2010) are proposed. The description of damage states for waterfront structures is provided in Table 4.6, while the corresponding vulnerability curves are given in Table 4.7 and Fig. 4.5.

Table 4.6 Description of damage states for waterfront structures subject to ground shaking

Damage state	Normalized residual hor. displ. (u_x/H)
Minor	Less than 1.5%
Moderate	1.5~5%
Extensive	5~10%
Complete	Larger than 10%

Table 4.7 Parameters of fragility curves for waterfront structures subject to ground shaking

	Median PGA (g) (rock outcrop conditions)			β (log-standard deviation)
	Minor damages	Moderate damages	Extensive damages	
$H \leq 10\text{m}$, $V_s = 250\text{m/s}$	0.11	0.37	0.81	0.54
$H \leq 10\text{m}$, $V_s = 500\text{m/s}$	0.07	0.34	-	0.58
$H > 10\text{m}$, $V_s = 250\text{m/s}$	0.14	0.44	0.96	0.49
$H > 10\text{m}$, $V_s = 500\text{m/s}$	0.10	0.40	-	0.57



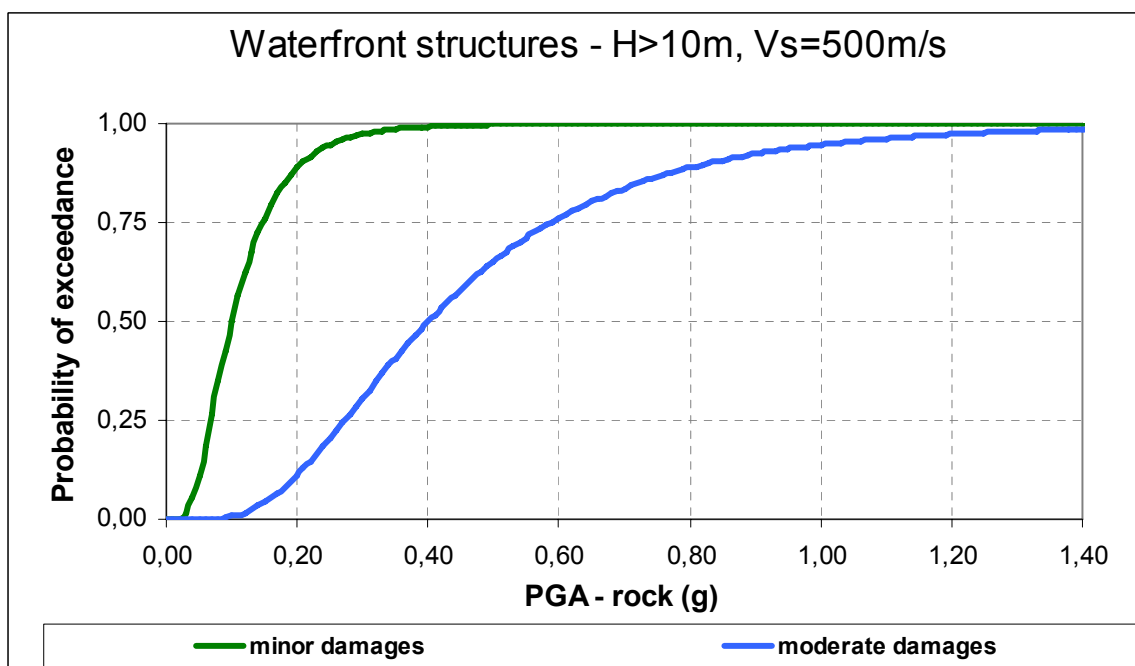
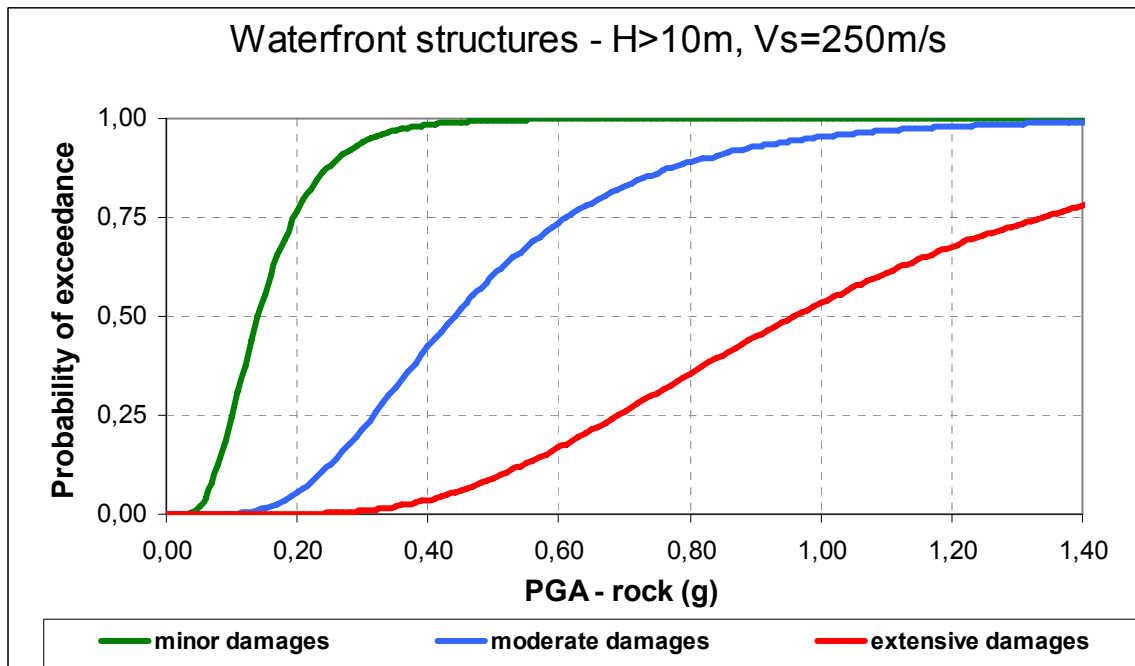


Fig. 4.5 Fragility curves for waterfront structures subject to ground shaking

4.3.2 Cargo handling and storage components

The HAZUS (NIBS, 2004) fragility curves are proposed for the vulnerability assessment of cargo handling and storage components.

The description of damage states for cargo handling and storage components is provided in Table 4.8, while the corresponding vulnerability curves are given in Table 4.9, Fig. 4.6 and Fig. 4.7.

Table 4.8 Description of damage states for cargo handling and storage components subject to ground shaking and ground failure

Damage state	Description		Serviceability	
	Stationary equipment	Unanchored or rail mounted equipment		
Minor	Slight damage to structural members with no loss of function	Minor derailment or misalignment without any major structural damage to the rail mount. Minor repair and adjustments may be required before the crane becomes operable.	Reduced use	Operational without repair
Moderate	Derailment due to differential displacement of parallel track. Rail repair and some repair to structural members is required.		Not usable	Operational after repairs
Extensive/ Complete	Considerable damage to equipment. Topped or totally derailed cranes are likely to occur. Replacement of structural members is required.			

Table 4.9 Parameters of fragility curves for cargo handling and storage components subject to ground shaking and ground failure

Description	Damage state	Peak Ground Acceleration (PGA)		Permanent Ground Displacement (PGD)	
		Median (g)	β	Median (m)	β
			(log-standard deviation)		(log-standard deviation)
Stationary equipment	Minor	0.30	0.60	0.08	0.60
	Moderate	0.50	0.60	0.15	0.70
	Extensive/ Complete	1.00	0.70	0.30	0.70
Unanchored or rail mounted equipment	Minor	0.15	0.60	0.05	0.60
	Moderate	0.35	0.60	0.10	0.60
	Extensive/ Complete	0.80	0.70	0.25	0.70

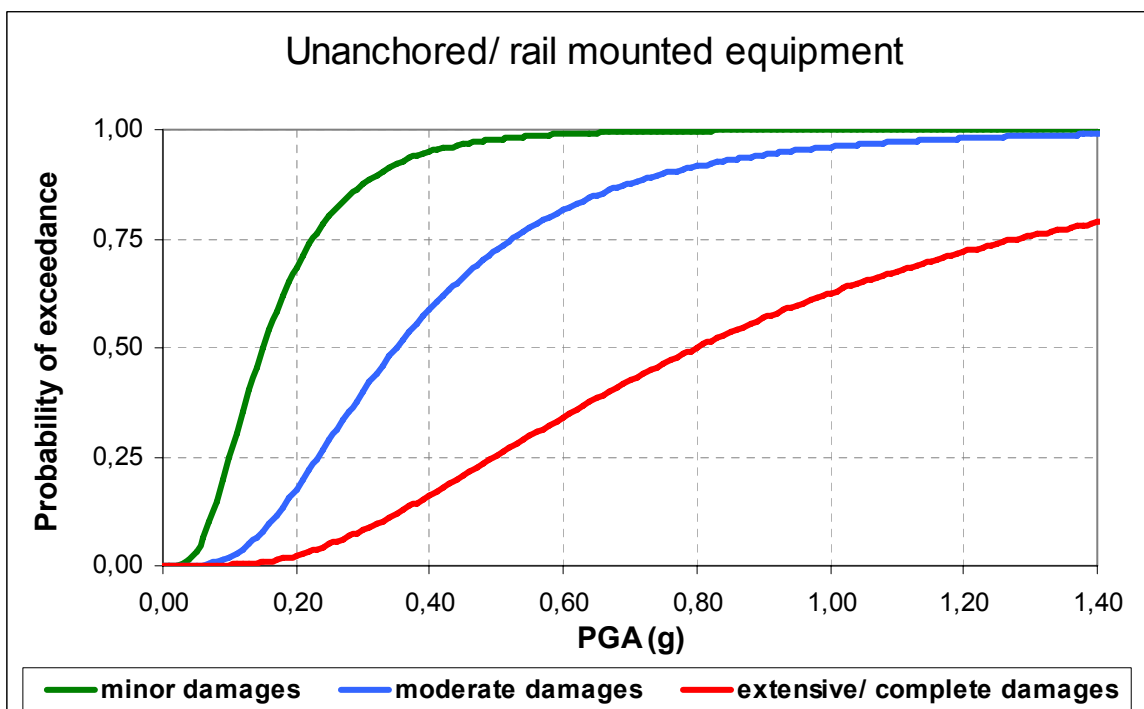
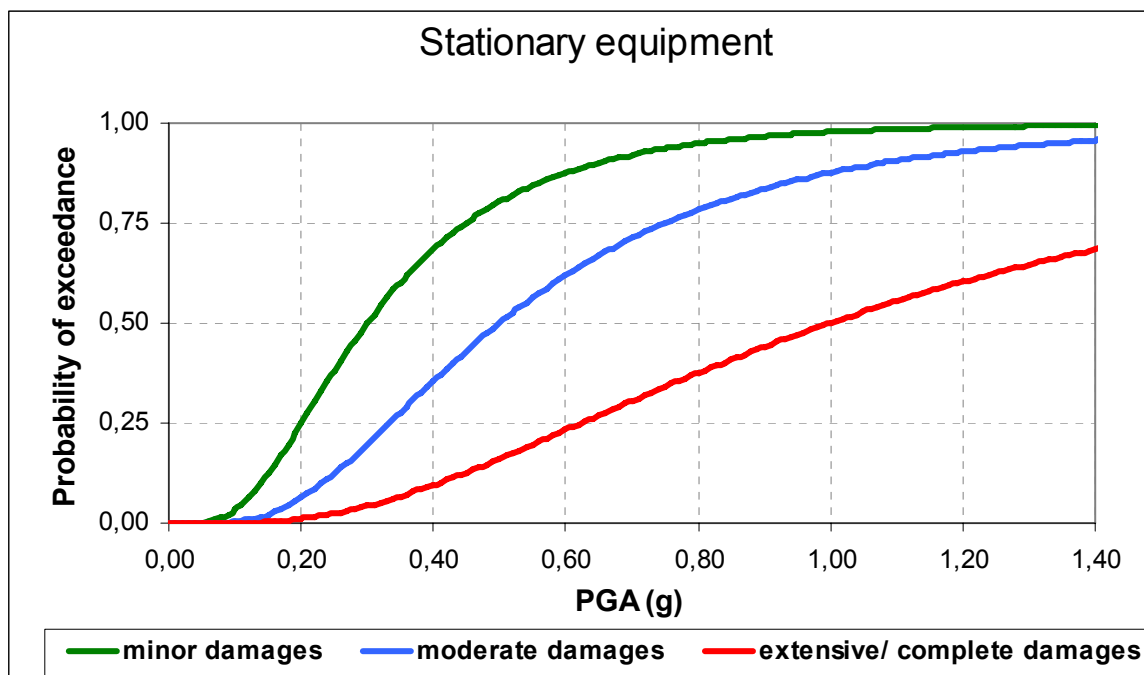


Fig. 4.6 Fragility curves for cargo handling and storage components subject to ground shaking

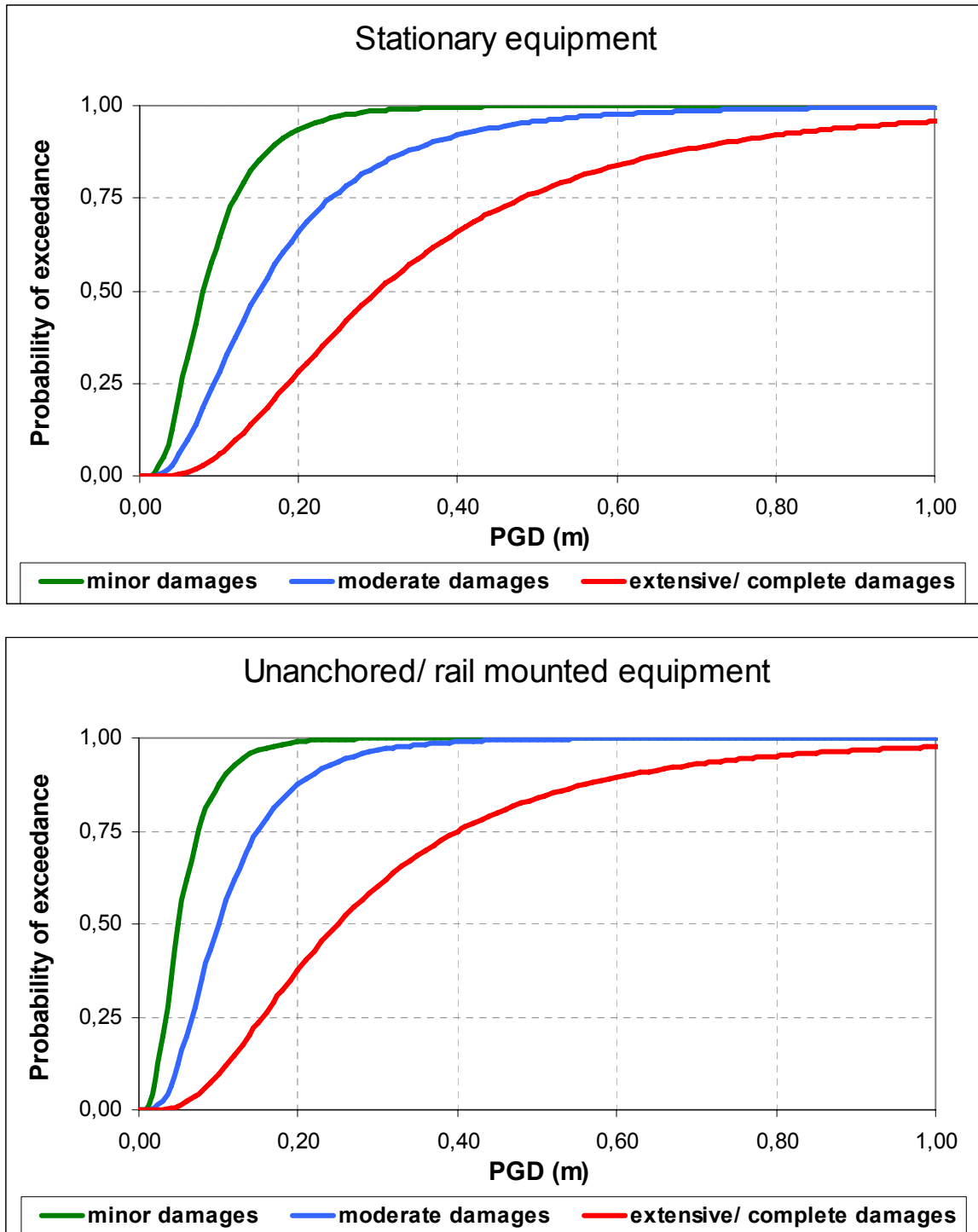


Fig. 4.7 Fragility curves for cargo handling and storage components subject to ground failure

4.3.3 Port infrastructures

Buildings

See D3.1 “Fragility functions for common RC building types in Europe” and D3.2 “Fragility functions for masonry buildings in Europe”.

Utility systems

Electric power system

See D3.3 “Fragility functions for electric power system elements”.

Water system

See D3.5 “Fragility functions for water and waste-water system elements”.

Waste-water system

See D3.5 “Fragility functions for water and waste-water system elements”.

Natural gas system

See D3.4 “Fragility functions for gas and oil network elements”.

Liquid fuel system

The SRMLIFE (2003-2007) fragility curves are proposed for the vulnerability assessment of liquid fuel facilities.

The description of damage states for fuel facilities is provided in Table 4.10 and Table 4.11, while the corresponding vulnerability curves are given in Table 4.12 and Table 4.13. Fig. 4.8 and Fig. 4.9 illustrate the corresponding fragility curves.

Comment: For the buildings sub-components, the typology and fragility curves proposed in SRM-LIFE (2003-2007) were used. The upgrade of fragility curves will be made after the finalization of D3.1 “Fragility functions for common RC building types in Europe” and the proposal of buildings’ typologies and fragility functions for SYNER-G.

Table 4.10 Description of damage states for fuel facilities subject to ground shaking

Damage state	Description		Serviceability	
	Anchored equipment	Unanchored equipment		
Minor	Slight damage to pump building, minor damage to anchor of tanks, or loss of off-site power (check electric power systems for more on this) for a very short period and minor damage to backup power (i.e. to diesel generators, if available).	Elephant foot buckling of tanks with no leakage or loss of contents, slight damage to pump building, or loss of commercial power for a very short period and minor damage to backup power (i.e. to diesel generators, if available).	Reduced use	Operational without repair
Moderate	Elephant foot buckling of tanks with no leakage or loss of contents, considerable damage to equipment, moderate damage to pump building, or loss of commercial power for few days and malfunction of backup power (i.e., diesel generators, if available).	Elephant foot buckling of tanks with partial loss of contents, moderate damage to pump building, loss of commercial power for few days and malfunction of backup power (i.e., diesel generators, if available).		Operational after repairs
Extensive	Elephant foot buckling of tanks with loss of contents, extensive damage to pumps (cracked/ sheared shafts), or extensive damage to pump building.	Weld failure at base of tank with loss of contents, extensive damage to pump building, or extensive damage to pumps (cracked/sheared shafts).	Not usable	Not repairable
Complete	Weld failure at base of tank with loss of contents, or extensive to complete damage to pump building.	Tearing of tank wall or implosion of tank (with total loss of content), or extensive/complete damage to pump building.		

Table 4.11 Description of damage states for fuel facilities subject to ground failure

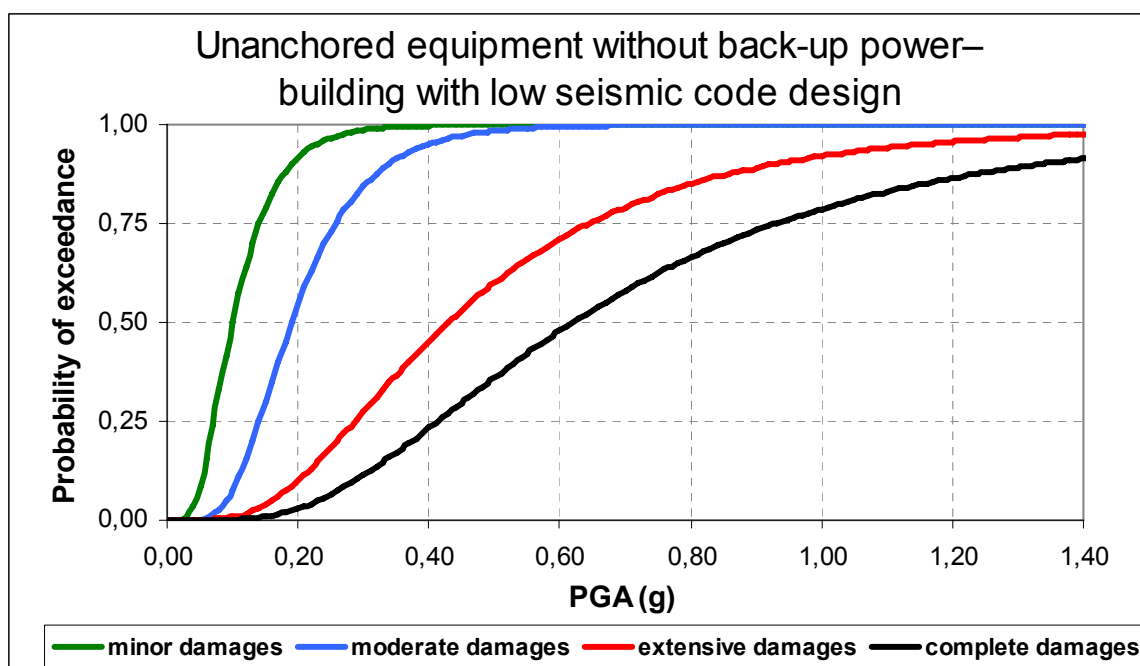
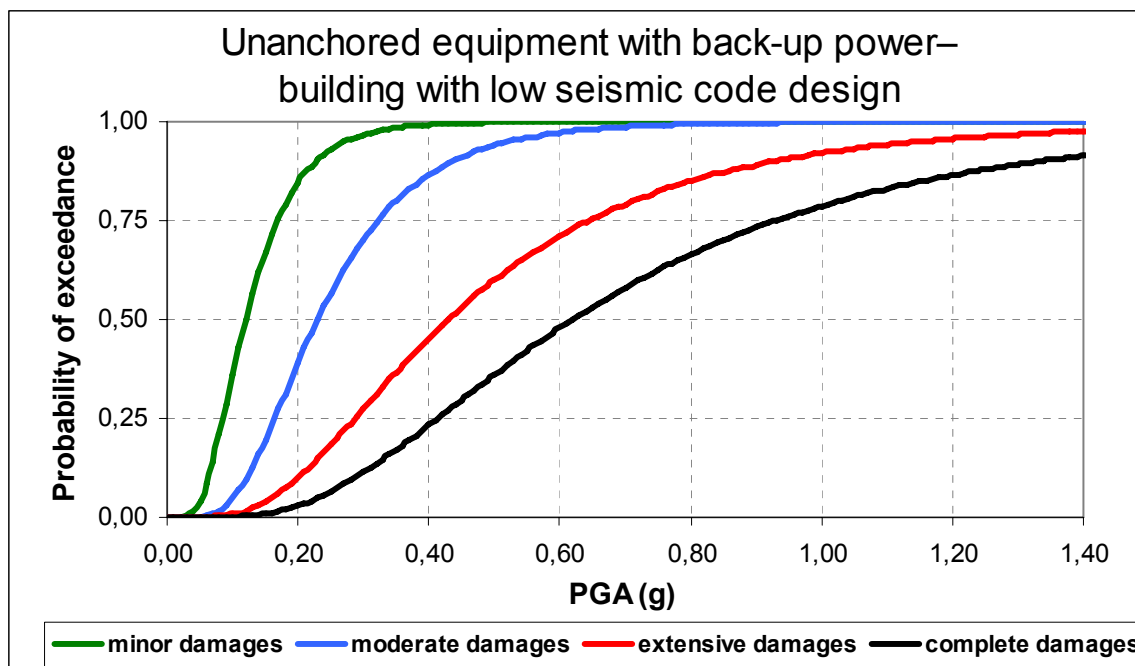
Damage state	Description		Serviceability	
	Buried tanks (PGD related damage)			
Minor	Minor uplift (few inches) of the buried tanks or minor cracking of concrete walls.		Reduced use	Operational without repair
Moderate	Damage to roof supporting columns, and considerable cracking of walls.		Not usable	Operational after repairs
Extensive/ Complete	Considerable uplift (more than 30cm) of the tanks and rupture of the attached piping.			Not repairable

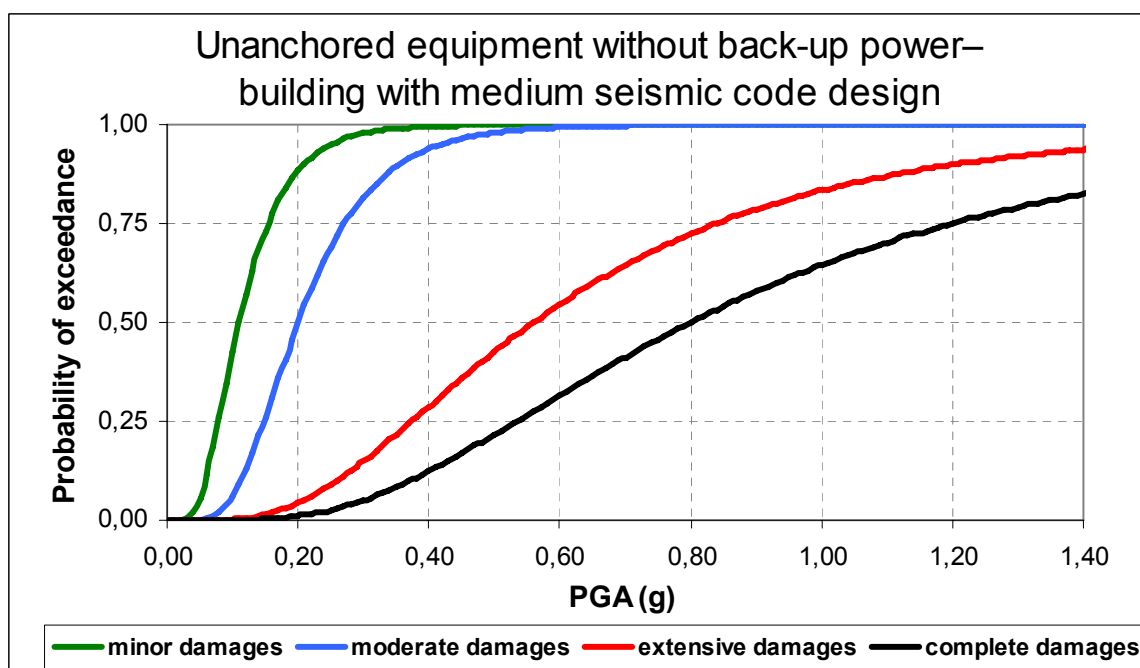
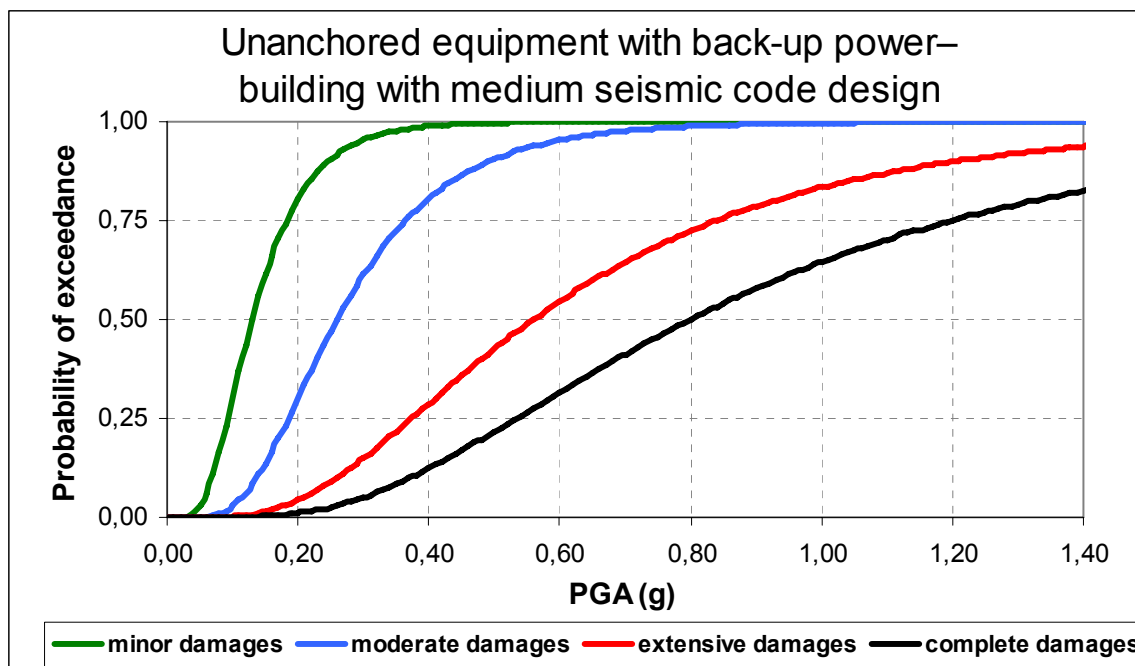
Table 4.12 Parameters of fragility curves for fuel facilities subject to ground shaking

Description	Damage state	Peak Ground Acceleration (PGA)	
		Median (g)	β (log-standard deviation)
Unanchored equipment with back-up power – building with low seismic code design	Minor	0.12	0.50
	Moderate	0.23	0.50
	Extensive	0.43	0.60
	Complete	0.62	0.60
Unanchored equipment without back-up power– building with low seismic code design	Minor	0.10	0.50
	Moderate	0.19	0.45
	Extensive	0.43	0.60
	Complete	0.62	0.60
Unanchored equipment with back-up power– building with medium seismic code design	Minor	0.13	0.50
	Moderate	0.26	0.50
	Extensive	0.56	0.60
	Complete	0.80	0.60
Unanchored equipment without back-up power– building with medium seismic code design	Minor	0.11	0.50
	Moderate	0.20	0.45
	Extensive	0.56	0.60
	Complete	0.80	0.60
Unanchored equipment with back-up power– building with high seismic code design	Minor	0.14	0.50
	Moderate	0.27	0.50
	Extensive	0.61	0.60
	Complete	0.90	0.60
Unanchored equipment without back-up power– building with high seismic code design	Minor	0.12	0.50
	Moderate	0.21	0.45
	Extensive	0.61	0.60
	Complete	0.90	0.60

Table 4.13 Parameters of fragility curves for fuel facilities subject to ground failure

Description	Damage state	Permanent Ground Displacement (PGD)	
		Median (m)	β (log-standard deviation)
Facilities with buried tanks	Minor	0.10	0.50
	Moderate	0.20	0.50
	Extensive/ Complete	0.61	0.50





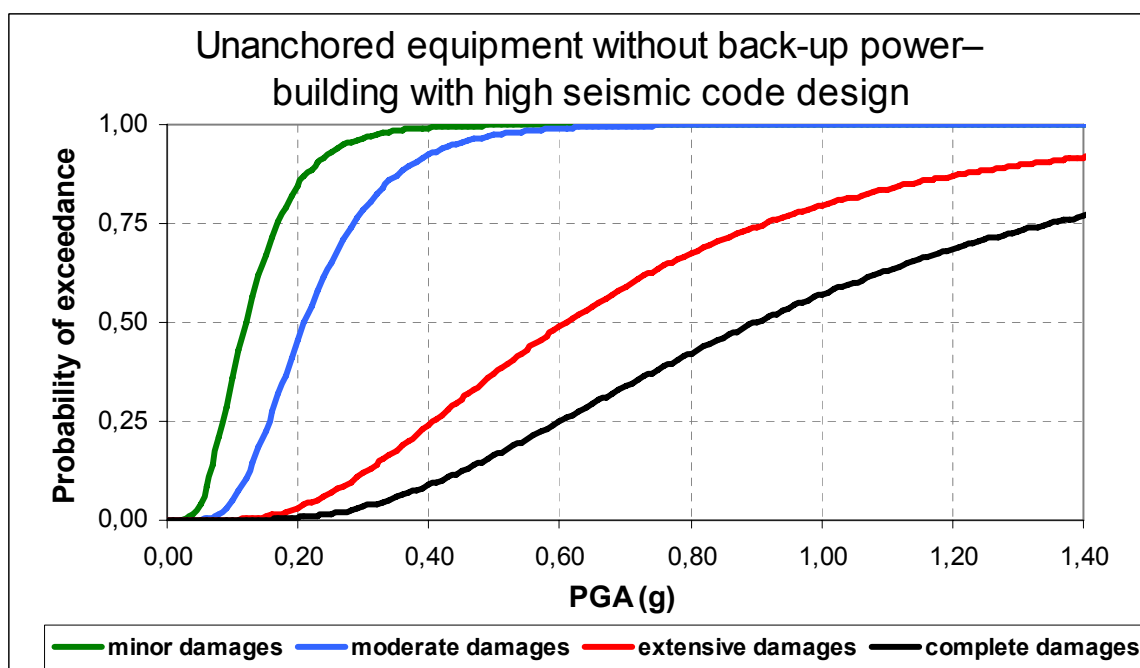
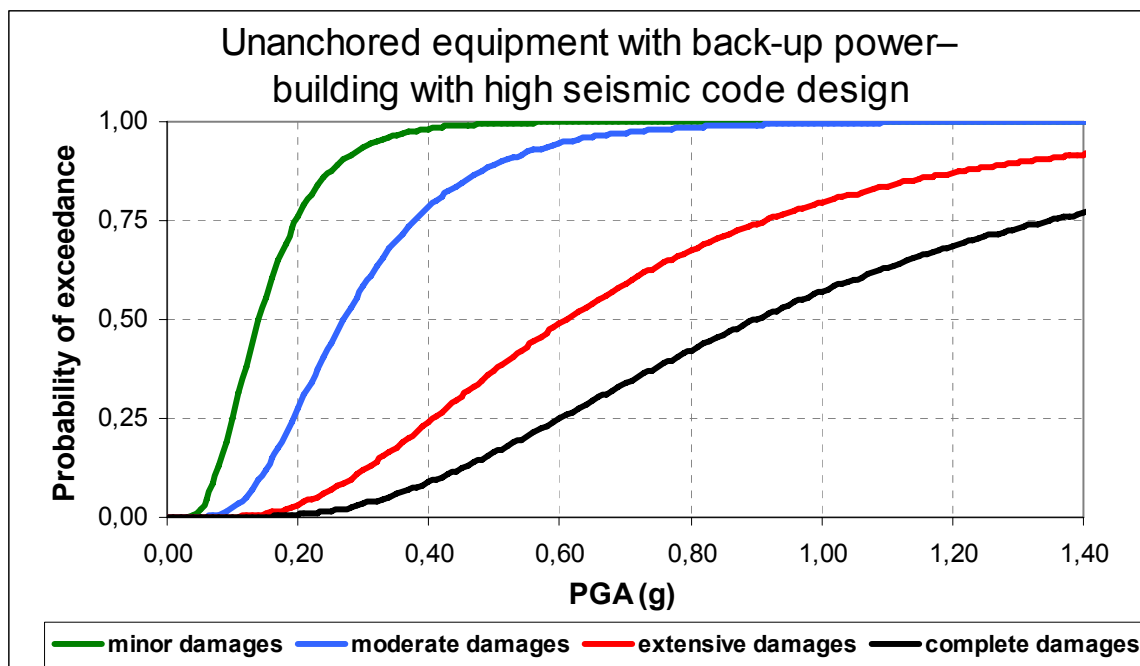


Fig. 4.8 Fragility curves for fuel facilities subject to ground shaking

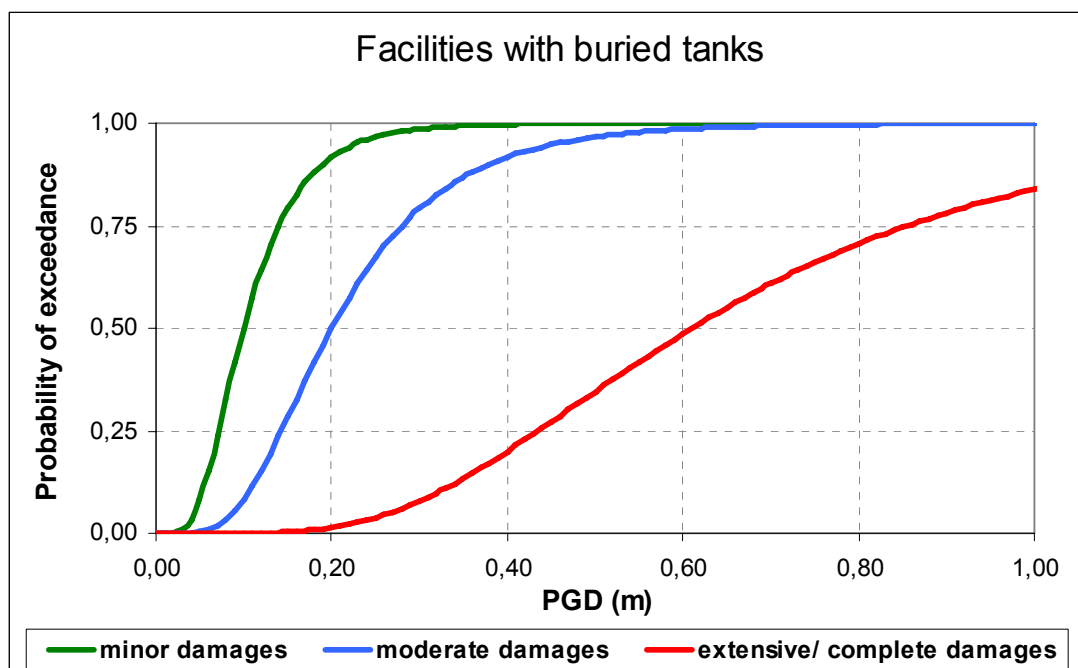


Fig. 4.9 Fragility curves for fuel facilities subject to ground failure

See also D3.4 "Fragility functions for gas and oil network elements".

Communications system

The SRMLIFE (2003-2007) fragility curves are proposed for the vulnerability assessment of communication facilities.

The description of damage states for communication facilities is provided in Table 4.14, while the corresponding vulnerability curves are given in Table 4.15 and Fig. 4.10.

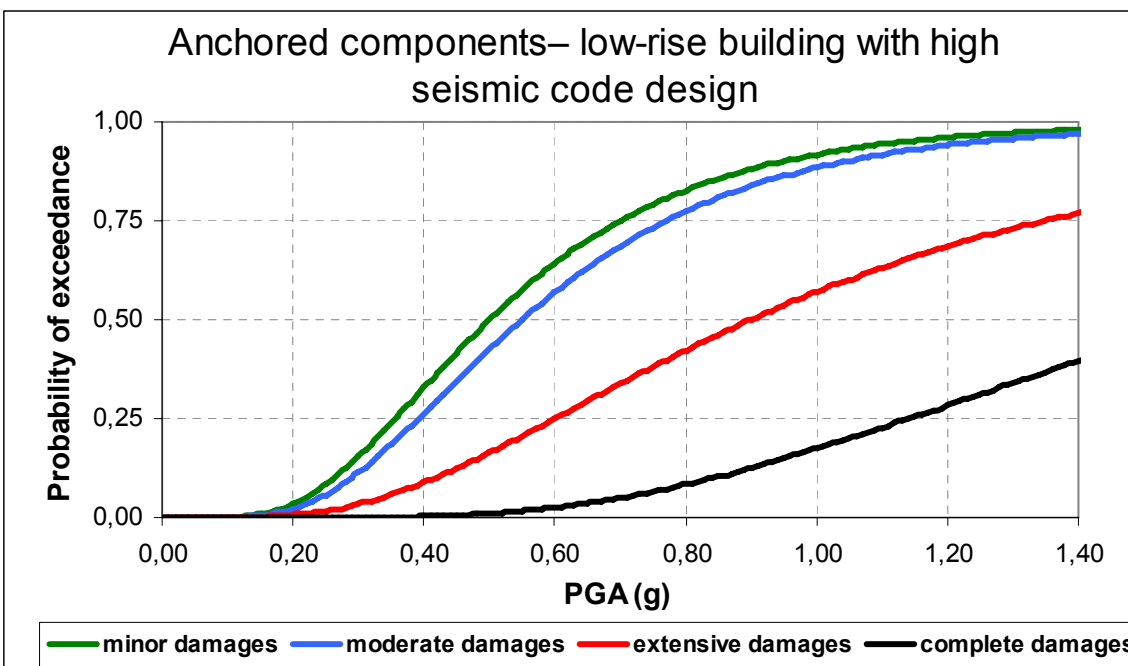
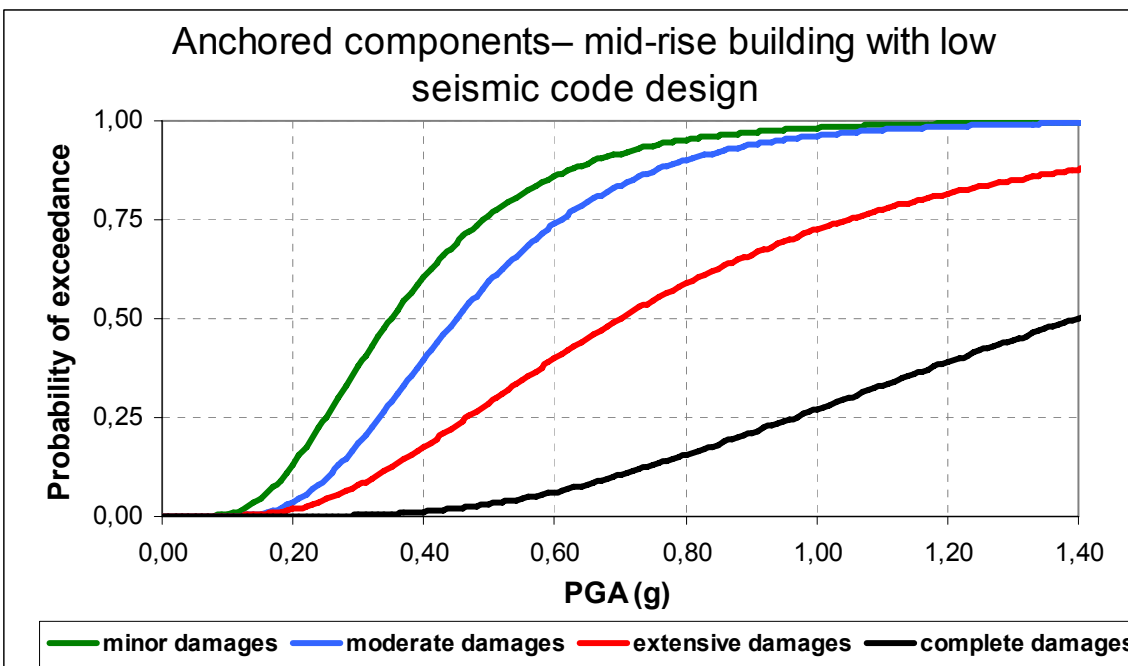
Comment: For the buildings sub-components, the typology and fragility curves proposed in SRM-LIFE (2003-2007) were used. The upgrade of fragility curves will be made after the finalization of D3.1 "Fragility functions for common RC building types in Europe" and the proposal of buildings' typologies and fragility functions for SYNER-G.

Table 4.14 Description of damage states for communication facilities

Damage state	Description	Restoration cost (%)	Serviceability	
Minor	Slight damage to the communication facility building, or inability of the center to provide services during a short period (few days) due to loss of electric power and backup power, if available.	10-30	Fully usable after limited repairs	Operational after limited repairs
Moderate	Moderate damage to the communication facility building, few digital switching boards being dislodged, or the central office being out of service for a few days due to loss of electric power (i.e., power failure) and backup power (typically due to overload), if available.	30-50	Fully usable after repairs	Operational after repairs
Extensive	Severe damage to the communication facility building resulting in limited access to facility, or by many digital switching boards being dislodged, resulting in malfunction.	50-75	Partially usable after extensive repairs	Partially operational after extensive repairs
Complete	Complete damage to the communication facility building, or damage beyond repair to digital switching boards.	75-100	Not usable	Not repairable

Table 4.15 Parameters of fragility curves for communication facilities

Description	Damage state	Peak Ground Acceleration (PGA)	
		Median (g)	β (log-standard deviation)
Anchored components— low-rise building with low seismic code design	Minor	0.30	0.55
	Moderate	0.40	0.60
	Extensive	0.55	0.60
	Complete	0.90	0.70
Anchored components— mid-rise building with low seismic code design	Minor	0.35	0.50
	Moderate	0.45	0.45
	Extensive	0.70	0.60
	Complete	1.40	0.55
Anchored components— low-rise building with high seismic code design	Minor	0.50	0.50
	Moderate	0.55	0.50
	Extensive	0.90	0.60
	Complete	1.60	0.50
Anchored components— mid-rise building with high seismic code design	Minor	0.35	0.55
	Moderate	0.55	0.45
	Extensive	0.85	0.55
	Complete	1.80	0.50



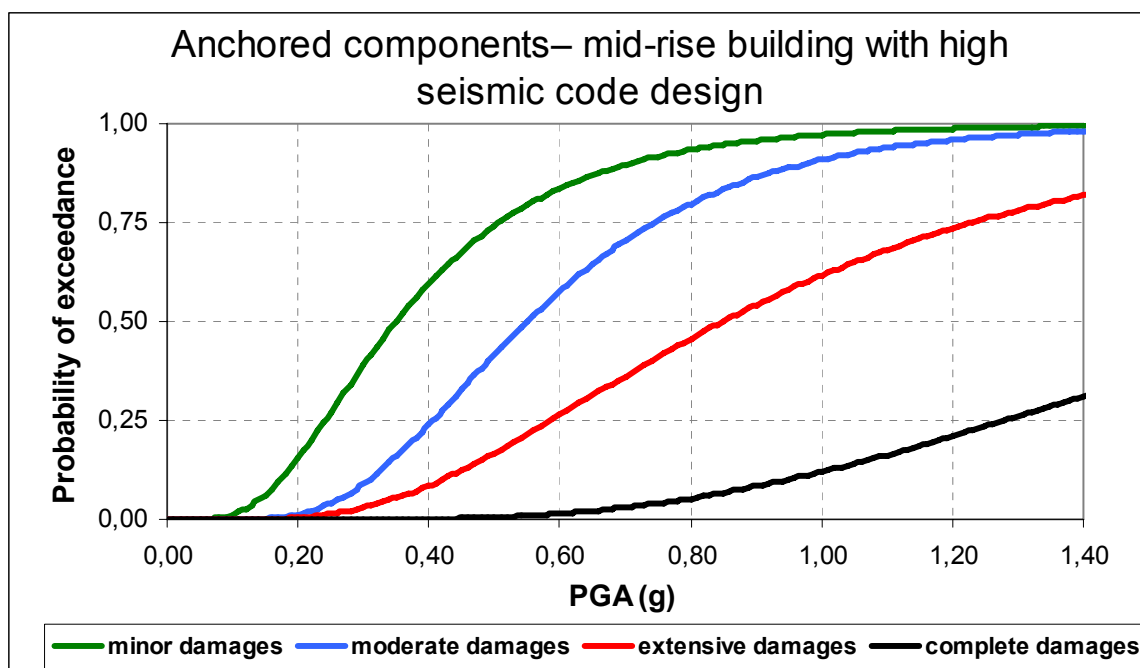


Fig. 4.10 Fragility curves for communication facilities

Fire-fighting system

See D3.11 "Fragility functions for fire fighting system elements".

Transportation infrastructures

Roadway system

See D3.7 "Fragility functions for roadway system elements".

Railway system

See D3.8 "Fragility functions for railway system elements".

Bridges

See D3.6 "Fragility functions for roadway bridges".

5 Coding and digital description of fragility functions

System	Harbor HBR																					
Element at risk	Waterfront structures	Code		HBR1a																		
Reference	NIBS, 2004																					
Method	Empirical																					
Function	Lognormal																					
Typology	-																					
Damage states	None	Minor	Moderate	Extensive	Complete																	
	-	Minor ground settlement resulting in few piles (for piers/seawalls) getting broken and damaged. Cracks are formed on the surface of the wharf. Repair may be needed.	Considerable ground settlement with several piles (for piers/seawalls) getting broken and damaged.	Failure of many piles, extensive sliding of piers, and significant ground settlement causing extensive cracking of pavements.	Failure of most piles due to significant ground settlement. Extensive damage is widespread at the port facility.																	
Functionality states	Usable	Reduced use Operational without repair	Not usable Operational after repairs	Not usable Not repairable																		
Seismic intensity parameter	Permanent Ground Displacement PGD (m)																					
Figures																						
Parameters (median values, β values)	<table border="1"> <thead> <tr> <th rowspan="2">Damage state</th> <th colspan="2">Permanent Ground Displacement (PGD)</th> </tr> <tr> <th>Median (m)</th> <th>β (log-standard deviation)</th> </tr> </thead> <tbody> <tr> <td>Minor</td> <td>0.13</td> <td>0.50</td> </tr> <tr> <td>Moderate</td> <td>0.30</td> <td>0.50</td> </tr> <tr> <td>Extensive</td> <td>0.43</td> <td>0.50</td> </tr> <tr> <td>Complete</td> <td>1.09</td> <td>0.50</td> </tr> </tbody> </table>					Damage state	Permanent Ground Displacement (PGD)		Median (m)	β (log-standard deviation)	Minor	0.13	0.50	Moderate	0.30	0.50	Extensive	0.43	0.50	Complete	1.09	0.50
Damage state	Permanent Ground Displacement (PGD)																					
	Median (m)	β (log-standard deviation)																				
Minor	0.13	0.50																				
Moderate	0.30	0.50																				
Extensive	0.43	0.50																				
Complete	1.09	0.50																				
Comments	No distinction of the structures typology																					

D3.9 - Fragility functions for harbor elements

System	Harbor HBR				
Element at risk	Waterfront structures	Code		HBR1b	
Reference	Kakderi and Pitilakis, 2010				
Method	Analytical				
Function	Lognormal				
Typology	Four typological classes: $H \leq 10\text{m}$, $V_s = 250\text{m/s}$ $H \leq 10\text{m}$, $V_s = 500\text{m/s}$ $H > 10\text{m}$, $V_s = 250\text{m/s}$ $H > 10\text{m}$, $V_s = 500\text{m/s}$				
Damage states	None	Minor	Moderate	Extensive	Complete
	-	Normalized residual hor. displ. (u_x/H) less than 1.5%	Normalized residual hor. displ. (u_x/H) 1.5~5%	Normalized residual hor. displ. (u_x/H) 5~10%	Normalized residual hor. displ. (u_x/H) larger than 10%
Functionality states	Usable	Reduced use Operational without repair	Not usable Operational after repairs	Not usable Not repairable	
Seismic intensity parameter	Peak Ground Acceleration PGA (g) (rock outcrop conditions)				
Figures					
Parameters (median values, β values)	Median PGA (g) (rock outcrop conditions)				β (log-standard deviation)
	Minor damages	Moderate damages	Extensive damages		
	$H \leq 10\text{m}$, $V_s = 250\text{m/s}$	0.11	0.37	0.81	0.54
	$H \leq 10\text{m}$, $V_s = 500\text{m/s}$	0.07	0.34	-	0.58
	$H > 10\text{m}$, $V_s = 250\text{m/s}$	0.14	0.44	0.96	0.49
	$H > 10\text{m}$, $V_s = 500\text{m/s}$	0.10	0.40	-	0.57
Comments	-				

D3.9 - Fragility functions for harbor elements

System	Harbor HBR																													
Element at risk	Cargo handling and storage components	Code	HBR2a																											
Reference	NIBS, 2004																													
Method	Empirical																													
Function	Lognormal																													
Typology	Two typological classes: Stationary equipment Unanchored or rail mounted equipment																													
Damage states	None	Minor	Moderate	Extensive/ Complete																										
	-	Stationary equipment: Slight damage to structural members with no loss of function Unanchored or rail mounted equipment: Minor derailment or misalignment without any major structural damage to the rail mount. Minor repair and adjustments may be required before the crane becomes operable.	Derailment due to differential displacement of parallel track. Rail repair and some repair to structural members is required.	Considerable damage to equipment. Topped or totally derailed cranes are likely to occur. Replacement of structural members is required.																										
Functionality states	Usable	Reduced use Operational without repair	Not usable Operational after repairs	Not usable Not repairable																										
Seismic intensity parameter	Peak Ground Acceleration PGA (g)																													
Figures																														
Parameters (median values, β values)	<table border="1"> <thead> <tr> <th rowspan="2">Description</th> <th rowspan="2">Damage state</th> <th colspan="2">Peak Ground Acceleration (PGA)</th> </tr> <tr> <th>Median (g)</th> <th>β (log-standard deviation)</th> </tr> </thead> <tbody> <tr> <td rowspan="3">Stationary equipment</td> <td>Minor</td> <td>0.30</td> <td>0.60</td> </tr> <tr> <td>Moderate</td> <td>0.50</td> <td>0.60</td> </tr> <tr> <td>Extensive/ Complete</td> <td>1.00</td> <td>0.70</td> </tr> <tr> <td rowspan="3">Unanchored or rail mounted equipment</td> <td>Minor</td> <td>0.15</td> <td>0.60</td> </tr> <tr> <td>Moderate</td> <td>0.35</td> <td>0.60</td> </tr> <tr> <td>Extensive/ Complete</td> <td>0.80</td> <td>0.70</td> </tr> </tbody> </table>				Description	Damage state	Peak Ground Acceleration (PGA)		Median (g)	β (log-standard deviation)	Stationary equipment	Minor	0.30	0.60	Moderate	0.50	0.60	Extensive/ Complete	1.00	0.70	Unanchored or rail mounted equipment	Minor	0.15	0.60	Moderate	0.35	0.60	Extensive/ Complete	0.80	0.70
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	Extensive/ Complete	0.80	0.70																											
Comments	-																													

D3.9 - Fragility functions for harbor elements

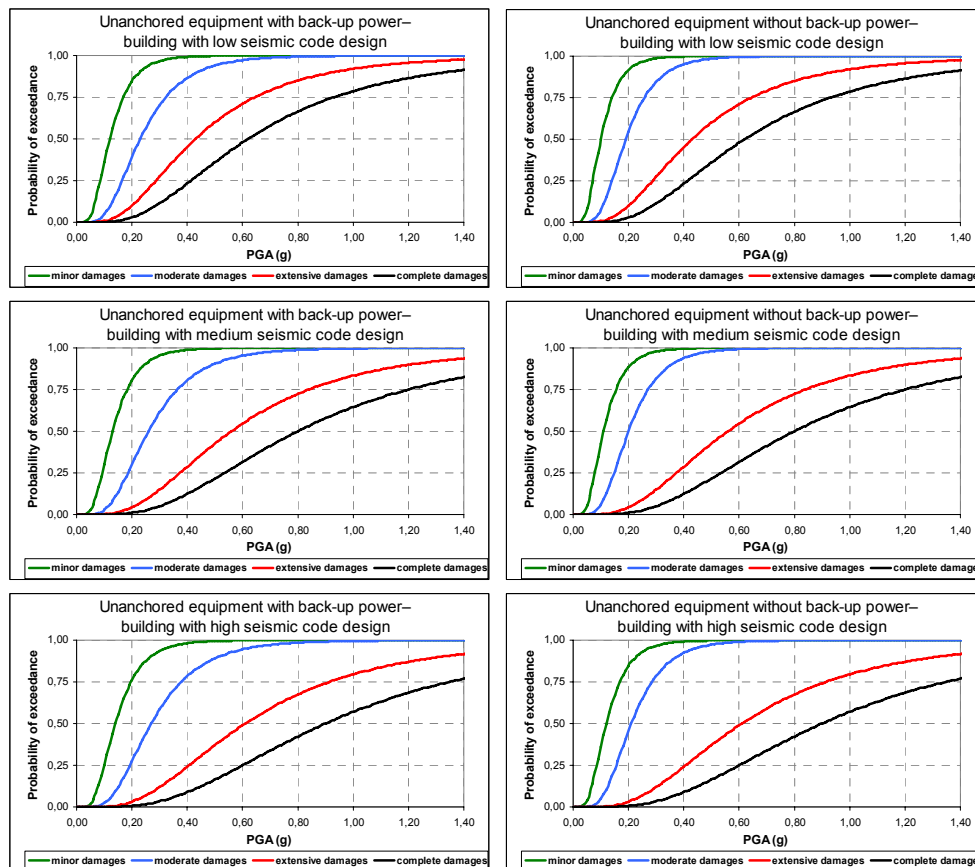
System	Harbor HBR																													
Element at risk	Cargo handling and storage components	Code	HBR2b																											
Reference	NIBS, 2004																													
Method	Empirical																													
Function	Lognormal																													
Typology	Two typological classes: Stationary equipment Unanchored or rail mounted equipment																													
Damage states	None	Minor	Moderate	Extensive/ Complete																										
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Seismic intensity parameter	Permanent Ground Displacement PGD (m)																													
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	Extensive/ Complete	0.25	0.70																											
Comments	-																													

D3.9 - Fragility functions for harbor elements

System	Harbor HBR				
Element at risk	Fuel facilities		Code	HBR3a	
Reference	SRMLIFE (2003-2007)				
Method	Empirical				
Function	Lognormal				
Typology	Two typological classes: Anchored equipment Unanchored equipment				
Damage states – Anchored equipment	None	Minor	Moderate	Extensive	Complete
	-	Slight damage to pump building, minor damage to anchor of tanks, or loss of off-site power (check electric power systems for more on this) for a very short period and minor damage to backup power (i.e. to diesel generators, if available).	Elephant foot buckling of tanks with no leakage or loss of contents, considerable damage to equipment, moderate damage to pump building, or loss of commercial power for few days and malfunction of backup power (i.e., diesel generators, if available).	Elephant foot buckling of tanks with loss of contents, extensive damage to pumps (cracked/sheared shafts), or extensive damage to pump building.	Weld failure at base of tank with loss of contents, or extensive damage to pump building.
Damage states – Unanchored equipment	-	Elephant foot buckling of tanks with no leakage or loss of contents, slight damage to pump building, or loss of commercial power for a very short period and minor damage to backup power (i.e. to diesel generators, if available).	Elephant foot buckling of tanks with partial loss of contents, moderate damage to pump building, loss of commercial power for few days and malfunction of backup power (i.e., diesel generators, if available).	Weld failure at base of tank with loss of contents, extensive damage to pump building, or extensive damage to pumps (cracked/sheared shafts).	Tearing of tank wall or implosion of tank (with total loss of content), or extensive /complete damage to pump building.
	Usable	Reduced use Operational without repair	Not usable Operational after repairs	Not usable Not repairable	
Seismic intensity parameter	Peak Ground Acceleration PGA (g)				

D3.9 - Fragility functions for harbor elements

Figures



Parameters (median values, β values)

Description	Damage state	Peak Ground Acceleration (PGA)	
		Median (g)	β (log-standard deviation)
Unanchored equipment with back-up power – building with low seismic code design	Minor	0.12	0.50
	Moderate	0.23	0.50
	Extensive	0.43	0.60
	Complete	0.62	0.60
Unanchored equipment without back-up power – building with low seismic code design	Minor	0.10	0.50
	Moderate	0.19	0.45
	Extensive	0.43	0.60
	Complete	0.62	0.60
Unanchored equipment with back-up power – building with medium seismic code design	Minor	0.13	0.50
	Moderate	0.26	0.50
	Extensive	0.56	0.60
	Complete	0.80	0.60
Unanchored equipment without back-up power – building with medium seismic code design	Minor	0.11	0.50
	Moderate	0.20	0.45
	Extensive	0.56	0.60
	Complete	0.80	0.60
Unanchored equipment with back-up power – building with high seismic code design	Minor	0.14	0.50
	Moderate	0.27	0.50
	Extensive	0.61	0.60
	Complete	0.90	0.60
Unanchored equipment without back-up power – building with high seismic code design	Minor	0.12	0.50
	Moderate	0.21	0.45
	Extensive	0.61	0.60
	Complete	0.90	0.60

Comments

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D3.9 - Fragility functions for harbor elements

System	Harbor HBR																			
Element at risk	Fuel facilities	Code	HBR3b																	
Reference	SRMLIFE (2003-2007)																			
Method	Empirical																			
Function	Lognormal																			
Typology	Two typological classes: Anchored equipment Unanchored equipment																			
Damage states	None	Minor	Moderate	Extensive/ Complete																
	-	Minor uplift (few inches) of the buried tanks or minor cracking of concrete walls.	Damage to roof supporting columns, and considerable cracking of walls.	Considerable uplift (more than 30cm) of the tanks and rupture of the attached piping.																
Functionality states	Usable	Reduced use Operational without repair	Not usable Operational after repairs	Not usable Not repairable																
Seismic intensity parameter	Permanent Ground Displacement PGD (m)																			
Figures																				
Parameters (median values, β values)	<table border="1"> <thead> <tr> <th rowspan="2">Description</th> <th rowspan="2">Damage state</th> <th colspan="2">Permanent Ground Displacement (PGD)</th> </tr> <tr> <th>Median (m)</th> <th>β (log-standard deviation)</th> </tr> </thead> <tbody> <tr> <td rowspan="3">Facilities with buried tanks</td> <td>Minor</td> <td>0.10</td> <td>0.50</td> </tr> <tr> <td>Moderate</td> <td>0.20</td> <td>0.50</td> </tr> <tr> <td>Extensive/ Complete</td> <td>0.61</td> <td>0.50</td> </tr> </tbody> </table>				Description	Damage state	Permanent Ground Displacement (PGD)		Median (m)	β (log-standard deviation)	Facilities with buried tanks	Minor	0.10	0.50	Moderate	0.20	0.50	Extensive/ Complete	0.61	0.50
Description	Damage state	Permanent Ground Displacement (PGD)																		
		Median (m)	β (log-standard deviation)																	
Facilities with buried tanks	Minor	0.10	0.50																	
	Moderate	0.20	0.50																	
	Extensive/ Complete	0.61	0.50																	
Comments	-																			

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