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Abstract

Critical infrastructures are the backbone of modern society and provide many essential goods and services. Recently, natural events impacting critical infrastructures have highlighted the vulnerability of these infrastructures to natural hazards. They have also revealed the risk of cascading failures with potentially major and extended societal and economic consequences.

This reports provides case-study descriptions of past incidents to shed light on the vulnerability of critical infrastructures to selected natural hazards. More specifically, incidents at refineries, large dams, hydrocarbon pipelines, natural gas storage and distribution, ports and industrial districts were analysed to better understand impact dynamics, system weaknesses, potential consequences and contributing factors. Based on the case histories and experience from similar incidents, lessons learned were derived for 1) system weaknesses and critical components, 2) potential for propagation, 3) consequence severity and extent and 4) protection systems and measures.

Keywords: Natural hazard, critical infrastructure, industrial accident, Natech

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1 Introduction

Critical infrastructures are the backbone of modern society and provide many essential goods and services, e.g. electrical power, telecommunications, water, etc. These infrastructures are highly integrated and often interdependent.

Recently, natural events impacting critical infrastructures have highlighted the vulnerability of these infrastructures to natural hazards. They have also revealed the risk of cascading failures with potentially major and extended societal and economic consequences. This risk is bound to increase in the future: On the one hand, global warming is already changing the severity and frequency of hydro-meteorological hazards, and growing industrialisation increases technological hazards. On the other hand, exposure and vulnerability are growing due to industry and community encroachment on natural-hazard prone areas and the increasing interconnectedness of society.

Given the risks associated with natural-hazard impact on critical infrastructures, the move towards a safer and more resilient society requires the development and application of an improved risk assessment framework to address high-impact-low-probability events. In this context, the FP7 project STREST (Harmonized approach to stress tests for critical infrastructures against natural hazards) aims at developing a stress test framework to determine the vulnerability and resilience of critical infrastructures. The developed stress-test methodologies will be applied to selected key European critical infrastructures in support of the implementation of European policies for the systematic application of stress tests.

Within the STREST project, Task 2.3 analyses the dynamics and consequences of recent critical events to derive systematic patterns of critical infrastructure disruption or failure and the potential for failure propagation for the purpose of learning lessons. Three principal critical-infrastructure classes were analysed with respect to the impact of earthquakes, tsunamis and floods:

a) Individual, single-site, high risk infrastructures,

b) Distributed and/or geographically extended infrastructures with potentially high economic and environmental impact,

c) Distributed multiple-site infrastructures with low individual impact but large collective impact or dependencies.

The following chapters provide summary descriptions of the dynamics and consequences of major past events for refineries, large dams, hydrocarbon pipelines, natural gas storage and distribution, ports and industrial districts. Case studies were selected based on natural hazards identified as relevant for the STREST project's six critical-infrastructure application sites. For every selected natural hazard and analysed critical infrastructure type, lessons learned were

identified for 1) system weaknesses and critical components, 2) potential for propagation, 3) consequence severity and extent and 4) protection systems and measures. These lessons learned are not only based on the case studies described in the report, but they also incorporate knowledge derived from experiences with other incidents of the same category.

It should be noted that with this study addressing six critical-infrastructure types, it is beyond its scope to produce detailed event descriptions or list all possible engineering or organisational solutions for better protecting a specific type of critical infrastructure. Rather, this work aims at highlighting the main structural and organisational weaknesses in critical infrastructures exposed to natural hazards to which attention should be paid. Ample references are provided for the interested reader who wishes to learn more about the vulnerability of specific infrastructures types.

2 Refineries and petrochemical facilities

2.1 EARTHQUAKES

2.1.1 Great East Japan (Tohoku) earthquake, Japan, 11 March 2011

The Great East Japan earthquake and the tsunami it triggered both reached unexpected magnitudes of severity and devastated an extended area. A recent report by the Japanese National Police Agency reported 15,884 deaths, 6,147 injured, and 2,636 people missing (NPAJ, 2014). In addition, about 400,000 buildings were confirmed as totally or half collapsed. With the total economic damages exceeding US$ 210 billion (not considering the Fukushima nuclear power plant accident), the Tohoku disaster is the most destructive on record (CRED, 2011).

2.1.1.1 *Trigger characteristics*

The Great East Japan earthquake occurred on 11 March 2011, at 14:46:24 local time. It was a subduction zone earthquake with M_W = 9.0 and it lasted about 200 seconds. The maximum peak ground acceleration (PGA) of 2.7g was recorded in Miyagi Prefecture (USGS, 2012). The source process with a large rupture distribution was on a fault plane of 450 by 150 km (JMA, 2011; Tsamba, 2012).

M = 8.2 tsunami earthquakes had been expected to occur with 20% probability in the coming 30 years in the region near the Japan trench between the northern Sanriku coast and the Boso Peninsula (Fujiwara and Morikawa, 2012). The Headquarters for Earthquake Research Promotion (HERP) had warned of the occurrence of an earthquake in offshore Miyagi Prefecture with a probability of 99% and M_{JMA} = 7.5 within 30 years. The event was expected to be similar to the Miyagi-oki earthquake (M_w = 7.6) of 1978. However, events corresponding to the 2011 Tohoku earthquake were not expected. The earthquake ruptured over several segments, which had been evaluated as independent earthquakes (Goto and Morikawa, 2012).

The Cosmo Oil refinery is located in Tokyo Bay (Chiba Prefecture) and was subjected to only low levels of earthquake forces. At the site of the refinery, PGA values of 0.114g were measured during the main shock and 0.099g during the aftershock (Krausmann and Cruz, 2013; Wada and Wakakura, 2011). The facility perimeter was not overtopped by the tsunami.

2.1.1.2 *Impact dynamics*

Damage and failure mechanisms

The Cosmo Oil refinery includes a storage depot for liquefied petroleum gas (LPG) which is a highly flammable substance. LPG storage tanks are big pressurised

spheres, held in place by legs supported by diagonal braces. Unknown to the refinery operator, the first earthquake shock damaged the braces on an LPG tank that at the time of the earthquake was filled with water for maintenance reasons. The second shock caused the weakened braces of the tank to buckle and subsequently the tank to collapse. This ruptured the LPG pipes connected to the tank and led to the release of flammable gases that eventually ignited (Cosmo Oil, 2011a; Krausmann and Cruz, 2013). Due to heat impingement by the fire on the other storage tanks, they ruptured via boiling liquid expanding vapour explosions (BLEVE). Overall, 5 major explosions were recorded.

Protection measures and systems

In principle, the LPG storage tanks were designed to withstand the earthquake forces they were subjected to. The tank that failed was filled with water, the additional weight of which had not been considered in the design stage. The design safety did therefore not take effect. In addition, the LPG pipes were equipped with an emergency valve that would automatically close in case of abnormal conditions. This valve had been manually switched open and could not be closed once the fire ignited. This demonstrates a clear failure of safety management oversight. These aspects are discussed in more detail in Section 2.1.1.4.

The automatic water-spraying systems used for external tank cooling continued to operate even after the evacuation of the fire fighters following the first explosion (Chiba Prefecture Fire Department, personal communication).

From an organisational safety management point of view, the refinery was not prepared for combating an event of this magnitude and had to rely on external help from different prefectures for dealing with the fires. With a major intervention both from the sea and on land, the last fires were extinguished 11 days after the earthquake.

2.1.1.3 Consequences

Damage to the critical infrastructure and its components

The refinery's LPG storage tank farm was completely destroyed, as well as surrounding pipelines and roads (Fig. 2.1). Adjacent asphalt tanks were damaged to due debris impact, and fires broke out in two neighbouring petrochemical facilities. In addition, damage to glass windows, vehicles, marine vessels and other structures was observed (Cosmo Oil, 2011a).

Socio-economic impact

The fires and explosions at the Cosmo Oil refinery led to the evacuation of 1,142 residents. At the refinery, 6 injuries (1 severe, 5 minor) were recorded while at a facility adjacent to the LPG tank farm, where a fire was triggered via a domino effect, 3 persons suffered injuries, one of them severe (Krausmann and Cruz, 2013). The

explosions were so violent that pieces of tank insulation and sheet metal were found at a distance of over 6 km from the refinery's tank farm. In nearby residential areas, windows and cars were damaged (Cosmo Oil, 2011a).

The fires at the refinery caused Cosmo Oil to declare a loss on disaster of over 5.7 billion ¥ (72 million US$[1]) for fiscal year 2010, which ended on 31 March, 2011 (Cosmo Oil, 2011b). For fiscal year 2011, the company reported a net deficit of 9.1 billion ¥ (114 million US$) which was mostly due to the suspension of activities at the Chiba refinery and associated alternative supply costs (Cosmo Oil, 2012). Overall, the refinery suffered a 2-year business interruption due to the damage caused by the Tohoku earthquake (Cosmo Oil, 2013).

Cascading effects

The fire caused by ignition of LPG from the broken pipes propagated to the other pressurised tanks which suffered BLEVEs. The explosions damaged nearby asphalt tanks due to debris impact, causing asphalt to leak into the ocean (Cosmo Oil, 2011a). In addition, dispersion of flammable LPG vapours and burning missile projection from the explosions started fires in two neighbouring chemical facilities, with subsequent hazardous-materials releases (Krausmann and Cruz, 2013).

Fig. 2.1 The LPG tank farm at the Chiba refinery after the earthquake-triggered fires and explosions (©2012 Google, ZENRIN)

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[1] Numbers in US$ were calculated based on the average 2011 Yen → US$ yearly exchange rate.

2.1.1.4   Contributing factors

The tank which collapsed was designed to withstand the earthquake forces it was subjected to assuming LPG filling. However, at the time of the earthquake it was filled with water which is 1.8 times heavier than LPG. It is believed that the additional weight caused the braces to crack when the first earthquake shock hit. Furthermore, it is good practice to leave water in the tanks for only 2-3 days during inspections. Nevertheless, the tank had already been filled with water for 12 days when the earthquake occurred, thus increasing the risk of an accident (Chiba Prefecture Fire Department, personal communication).

An emergency valve on the LPG pipes was manually switched to the "open" position in violation of safety regulations. Once the pipes broke due to the collapse of the tank they were connected to and the LPG ignited, the valve could not be reached and closed, thereby continuously feeding LPG to the fire. This is probably the single most important factor that caused the fire to burn out of control (Chiba Prefecture Fire Department, personal communication; Cosmo Oil, 2011a).

2.1.2   Kocaeli earthquake, Turkey, 17 August 1999

The Kocaeli area in Northwest Turkey has a population of 20 million inhabitants (one third or Turkey's total population) and encompasses nearly half of the country's industry. The Kocaeli earthquake was the first event of a sequence, followed by the Duzce earthquake on 12 November and is considered one of the most devastating earthquakes of the twentieth century, in view of the number of casualties and damage.

Several critical structures were severely damaged by the earthquake. The highway and rail transport system was seriously compromised by seismic shaking, active crossing faults and induced landslides (Erdik, 2000). Many plants suffered malfunctioning and business interruption because they were located in the epicentral zone. According to the isoseismal maps of the earthquake, the most industrialized area of Turkey was within the boundaries of intensity VIII to X in terms of MMI (Tang, 2000). Several state-owned refining and petrochemical complexes are located within 5 km of the fault, including Tupras, Petkim and Igsas.

2.1.2.1   Trigger characteristics

On 17 August 1999 at 3:02 a.m. a M_w=7.4 earthquake with a focal depth of 17 km struck the Kocaeli area. The effective duration and PGA of accelerograms recorded by stations within 15 km distance from the surface expression of the fault ranges between 15 and 44 seconds and 0.15 - 0.4g, respectively (Sucuoğlu, 2002). The total duration of the earthquake was about 45 seconds (Özerdem and Barakat, 2000). The event was expected, considering the high and recurrent activity of the North Anatolian strike-slip fault. Twelve significant earthquakes ($M_w \geq 6.5$) occurred along the fault from 1939 to 1999 (Erdik et al. 2004). The earthquake was associated with a 120 km rupture involving four distinct fault segments of the northernmost

strand of the western extension of the 1,300 km long North Anatolian fault system. Predominantly, right-lateral strike slip offsets were in the range of 3-4 m over a significant length of the fault (Durukal and Erdik, 2008; Erdik and Durukal, 2008).

The Tupras refinery is located along the shore at Tütünçiftlik of the western Kocaeli province. The plant is owned by the state oil company and was designed and constructed in 1961. It produces naphtha, gasoline, jet-oil and kerosene. The location of the plant is shown in Fig. 2.2. Due to the refinery's vicinity to the Tupras fault, the response at the site was strongly affected by the near-source motion. Furthermore, at that location no ground failure occurred except some liquefaction at landfills during the earthquake. According to shakemap data from the US Geological Survey (USGS, 2014), the PGA at the Tupras site was about 0.4 g (USGS, 2014).
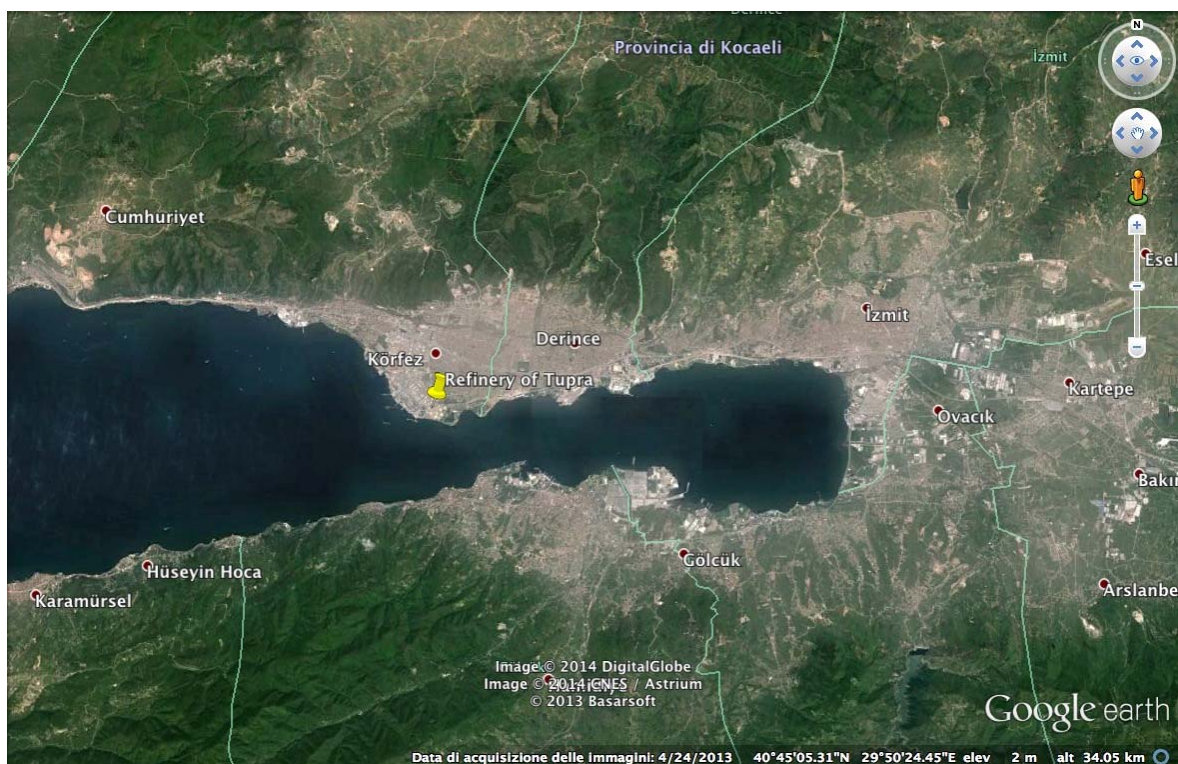
Fig. 2.2 Location of the refinery of Tupras

2.1.2.2 Impact dynamics

The Tupras plant is the largest refinery in this region of Turkey, accounting for about 1/3 of the national oil-related output, and it is a major supplier to much of the industry in the area. Being the 7th largest plant in Europe the annual production of processed oil is 12 million tons. Crude oil was stored in 14 large cylindrical tanks, semi-products were stored in 86 middle- to small size cylindrical tanks. Among the process plants affected by the earthquake, the Tupras refinery was the most severely damaged Fig. 2.3.

Fig. 2.3 Aerial view of the Tupras refinery after the Kocaeli earthquake

Damage and failure mechanisms

The post-earthquake damage observed at the Tupras refinery was surprisingly more extensive than that observed for other earthquakes with similar ground motion levels. This is due to the proximity to the earthquake epicenter (JSCE, 1999).

The sloshing of liquid in atmospheric storage tanks in the naphtha tank farm damaged tank perimeter seals and caused damage and releases from the tank tops (Fig. 2.4). The vertical movement of the floating roof against the metallic tank shell due to seismic loading created sparks which led to the ignition of the oil leakage (Erdik, 2000). Leaking naphtha from a damaged flange on one tank ignited, flowed downstream through the refinery's drainage system and spread the fire to two additional naphtha tanks (Steinberg et al, 2001).

Six cylindrical floating roof tanks were burned following damage by the earthquake. Four middle-sized naphtha tanks with a diameter of 20-25 m and two small-sized tanks with a diameter of 10 m were damaged as a result of thermal deformation (JSCE, 1999). Due to radiant heat from the fires in the tank farm, the fire also spread to a cooling tower (Fig. 2.4). The extreme heat caused substantial deformation and damage in approximately 20 steel tanks. Several other floating-roof tanks (about 46) of different diameter were also damaged by the earthquake, e.g. due to elephant foot buckling.

The earthquake loading triggered the collapse of a 115 m heater stack onto the boiler and crude oil processing unit. Parts of the collapsed stack directly hit an upper super heater and a pipe rack, and caused a second fire (Fig. 2.5 and Fig. 2.6). The stack failed because of a larger opening at the cross section (Kilic and Sozen, 2003; JSCE, 1999). Fig. 2.7 provides an overview of the location of the damaged tanks and fires at the refinery.

None of the pressurized steel spherical tanks suffered damage. Also pipelines were not affected by the seismic shaking even if some damage was observed due to the fires that ignited. No substantial sliding of the anchored tanks and no evidence for pipe failure in the rest of the tanks at the tank farm was observed.

Fig. 2.4 Tank fire at Tupras refinery due to collision of the floating roof with the tank wall (Courtesy of Tupras A.Ş. (TUPRAS, 2000))

a.   b.
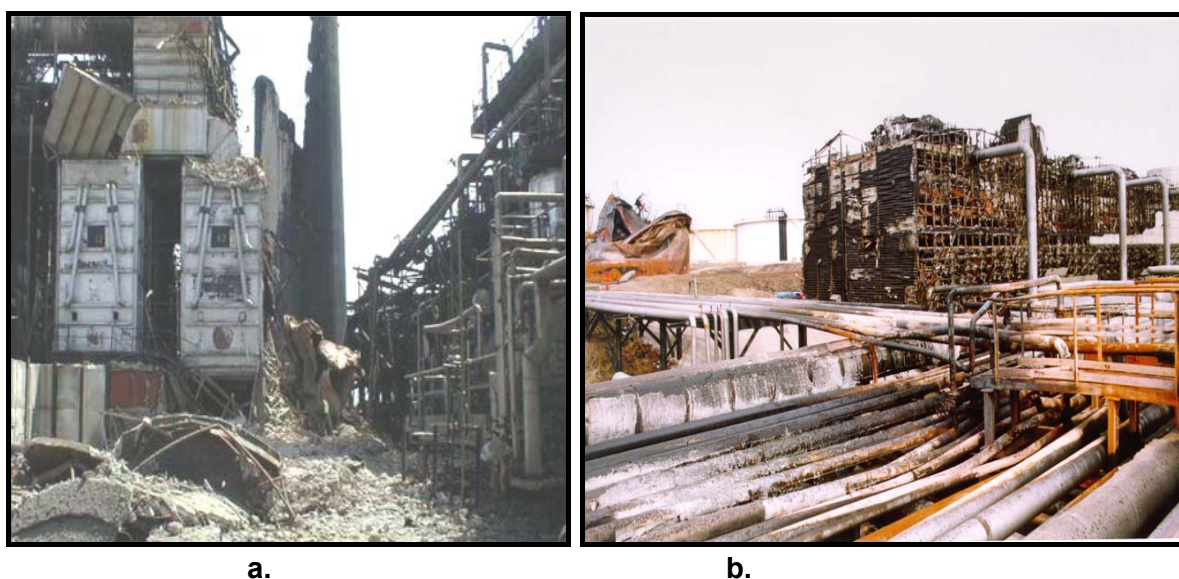
Fig. 2.5 Burned cooling tower (left) by the radiant heat of the burning tanks and damaged heater due to collapse of the stack (left) at TUPRAS refinery

Protection measures and systems

Most tanks were built in 1961 according to the earthquake design code of California for Level 4 earthquake shaking. It is evident that this level of design safety was not adequate to provide resistance against the seismic hazard of the area. According to Girgin (2011), the Tupras refinery was not prepared for the Natech events, in spite of the elevated earthquake hazard.
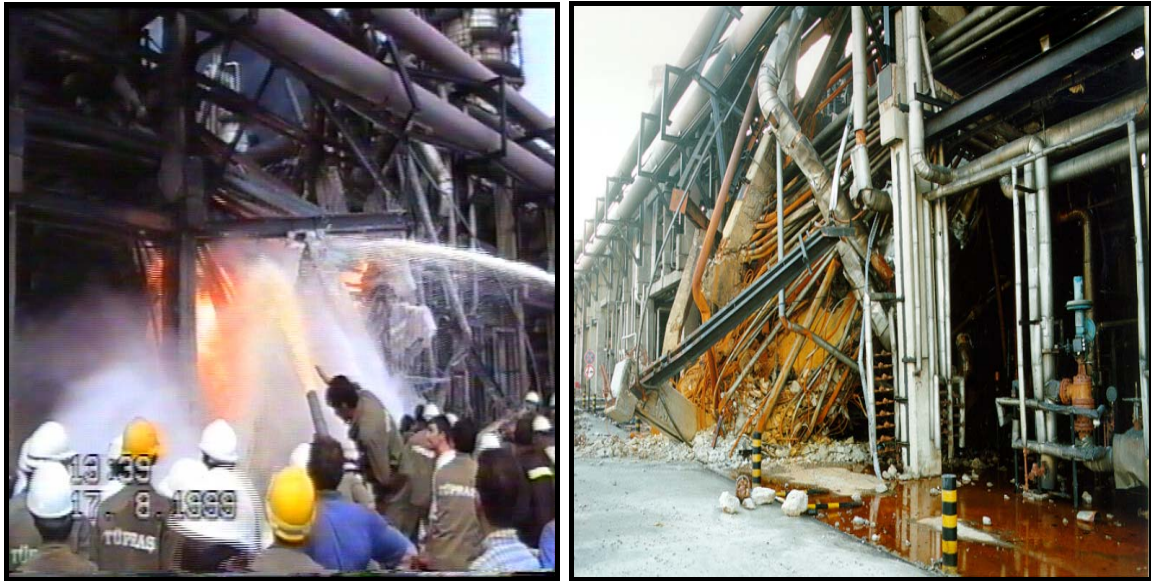
Fig. 2.6 Damage and fire at the pipe rack due to collapsed stack at TUPRAS refinery
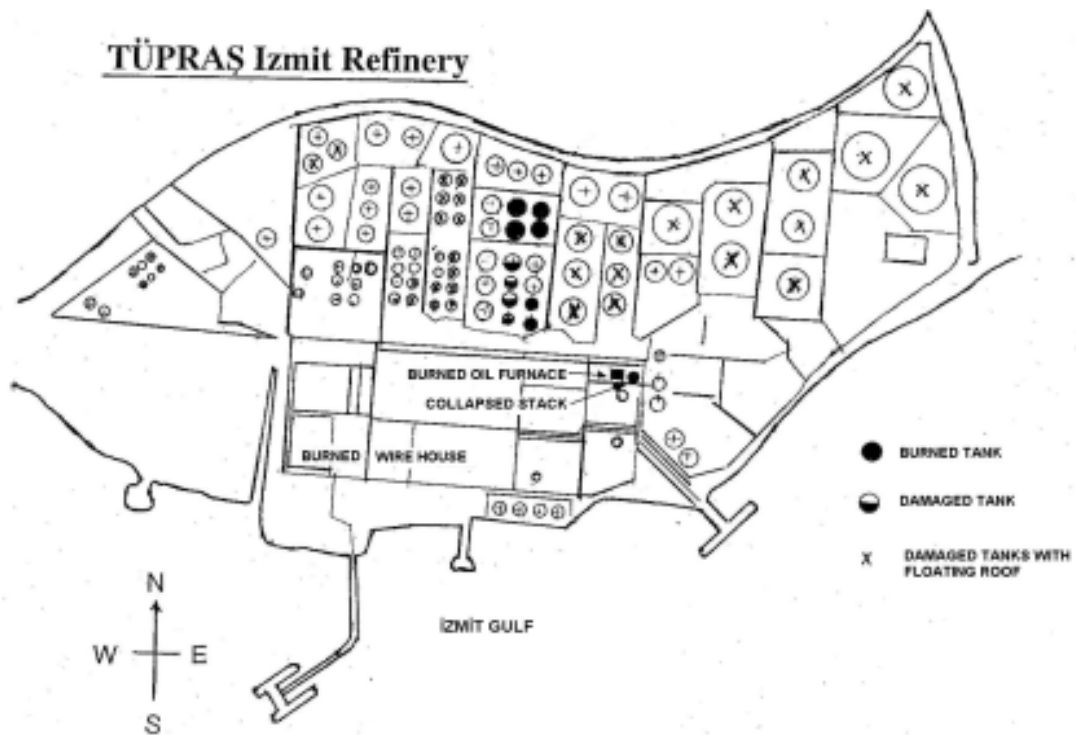
Fig. 2.7 Locations of damaged and burned tanks (JSCE, 1999)

2.1.2.3 Consequences

Damage to the critical infrastructure and its components

The earthquake caused significant structural damage to the refinery itself and the associated tank farm. It also damaged the crude oil and product jetties. The subsequent fires caused extensive additional damage. Overall, six steel tanks of

varying sizes in the tank farm of 112 tanks suffered complete collapse due to ground shaking and fire. Many more tanks were subject to deformation by heating or suffered structural damage. There was damage to cooling towers and the port area.

Socio-economic impact

Environmental pollution was reported in the whole region. With respect to economic impact on the operator, Turkish authorities stated that the total damage to Tupras refinery was worth about 0.5 billion US$. These damages were insured.

The facilities in the region hit by the earthquake are strongly interconnected via the supply of raw materials or products. The damage to Tupras refinery, together with the earthquake-triggered downtime at other key facilities, had a negative impact on many businesses in the area.

Cascading effects

The earthquake-triggered releases of flammable materials ignited and the fire propagated throughout the tank farm, causing extensive additional damage. The collapse of a heater stack created a secondary accident that ultimately also led to a fire. Due to insufficient and inoperable valves, the flow from the product lines could not be shut off (Steinberg et al., 2001).

The fire at the Tupras refinery risked to cause cascading effects in other areas of the refinery and in adjacent facilities. The day after the fire ignited, a barrier had to be erected between the burning naphtha tank farm and the refinery's LPG tanks to prevent the fire from reaching that part of the refinery and causing a BLEVE (Girgin, 2011). The fire also threatened to cascade to a nearby petrochemical facility and an adjacent fertilizer manufacturing facility. Due to the risk of explosions and hazardous-materials releases from all these facilities, authorities recommended the evacuation of a 5 km area around the refinery (Steinberg et al., 2001).

2.1.2.4 *Contributing factors*

Overloading of the emergency response due to the multiple and contemporary accidents seriously aggravated the damage propagation. While extinguishing the fire at the crude oil unit, the fire in the tank farm burned out of control. With the help of international relief efforts, the fire at the tank farm was completely extinguished only four days after the earthquake. Moreover, emergency-response operations were hampered because of the failure of civil infrastructures (roads, bridges, etc.), needed to reach the plant.

2.1.3 **Northridge Earthquake, USA, 17 January, 1994**

The Northridge earthquake in 1994 caused extensive damage to major lifeline facilities in the Los Angeles area. It was a triggering event for the development of methods and procedures for the seismic analysis of infrastructures. Electric power

transmission, water and natural gas distribution systems, telecommunication equipment, and a number of key highway bridges throughout the epicentral area. Several industrial plants were also damaged during the event, thus causing strong economic losses in San Fernando Valley, Santa Monica Valley, Burbank, Glendale, Santa Clarita Valley and Los Angeles.

The region affected by the 1994 Northridge earthquake is a prominent zone of high hazards, extending from south of Santa Barbara to the northern part of the San Fernando Valley and the bordering mountains, because of a group of strike-slip and thrust faults (also blind thrust faults similar to the one that produced the Northridge earthquake) with relatively high slip rates (USGS, 2013). The probability of a magnitude 7 or greater earthquake by the year 2024 is estimated as high as 80%-90% in southern California.

No relevant damage due to the Northridge earthquake was reported for oil refineries. However, power generation plants located near the epicentre were severely damaged. Since they include storage tanks and pipes similar to what is found at oil refineries, the relevant earthquake effects at a power plant are presented based on studies by Lau et al. (1995) and Schiff (1997).

2.1.3.1 Trigger characteristics

The Northridge earthquake occurred at 4:31 a.m. local time on 17 January 1994. The epicenter was located about 30 km west-northwest of downtown Los Angeles at a focal depth of 19 km (Murakami et al., 1996). The ruptured length of the predominantly thrusting fault was about 15 km whereas the total rupture duration was about 7 seconds (Wald et al., 1996). The National Earthquake Information Center calculated the 20-s surface wave magnitude of the earthquake as 6.7 (M_s) and the USGS assigned it a seismic moment magnitude (M_w) of 6.7 (Schiff, 1997).

Near the AES Pacerita electric power generating plant located 17 km north of the epicentre, the recorded peak ground accelerations were 0.63 g in the horizontal direction and 0.62 g in the vertical direction. The plant was built in 1985, and it was under normal maintenance shutdown at the time of the earthquake.

2.1.3.2 Impact dynamics

Damage and failure mechanisms

The earthquake caused damage to many liquid storage tanks in the power plant. Two 113,500 l welded steel tanks shifted 10 cm from their original positions, even though the tanks were anchored to a reinforced concrete ring foundation by 5.08 cm diameter solid steel anchor bolts. Both tanks suffered a permanent differential settlement of about 10 cm.

Pipes were extensively damaged due to shifting and rocking movements of heavy equipment and vibrations of the connecting pipes. Many pipes fractured and subsequently fell from their supports. In some cases, the use of flexible steel

reinforced connecting pipe allowed for the pipe to accommodate the deformation and stress imposed on it by the separate vibrations of the two connected pieces of equipment during the earthquake, and to sustain the permanent differential displacement remaining at the end of the strong ground motion.

Protection measures and systems

It is assumed that the plant was designed according to the Californian seismic code. For this reason, the damage to tanks could be considered a significant non-negligible issue.

2.1.3.3 Consequences

Damage to the critical infrastructure and its components

One of the fire fighting water tanks was a bolted steel tank unanchored at the base. The bolted tank failed in "elephant foot" buckling of the tank wall around its base, as shown in Fig. 2.8, whereas the welded tank anchoring system prevented any damage to the welded tank wall.
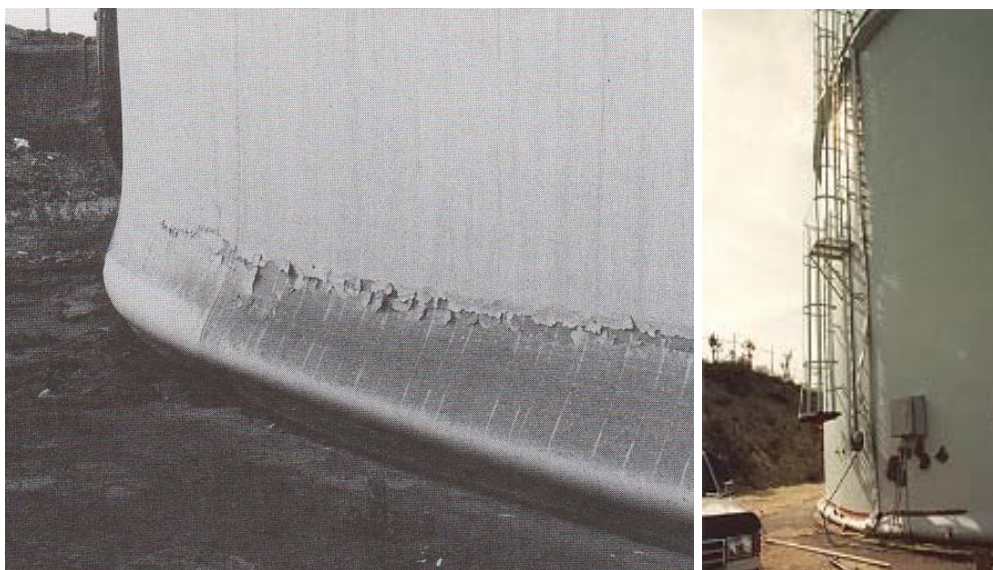
Fig. 2.8 Elephant foot buckling of a bolted steel tank

Deformations of the anchor bolts shown in Fig. 2.9 clearly indicated that the welded tank rocked significantly in response to the overturning moment generated by the hydrodynamic liquid pressure during the earthquake. The reinforced concrete foundation was damaged owing to pull-out of the anchor bolts. Although the anchorage system of the welded tank was badly damaged, it evidently saved the tank from more serious damage by preventing the tank from rocking off its foundation and the tank wall from buckling by the highly concentrated compressive axial stress resulting from the overturning rocking motion, as happened in the case of the bolted unanchored tank. After the earthquake, ground cracks of up to 45.7 cm

in width and 76.2 cm drop in elevation were noted in the surrounding area of the two tanks.
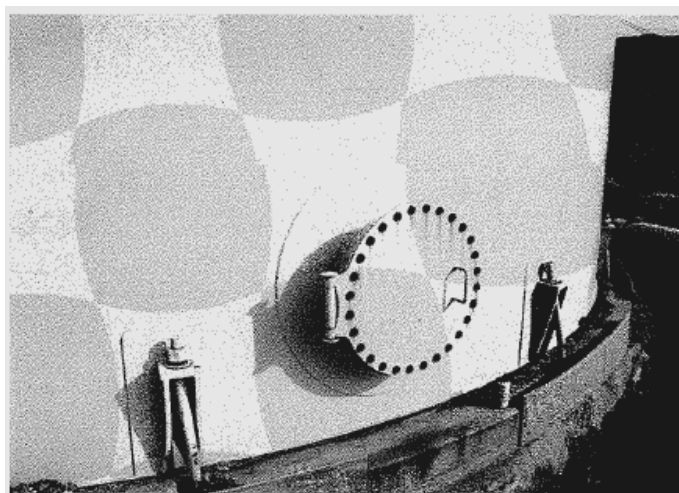
Fig. 2.9 Pulling out and twisting of the anchor bolts (Lau et al. 1995)

Socio-economic impact

Since the power plant was not in operation at the time the earthquake hit, no social impact is resulted. There is no information on the economic impact of the event.

Cascading effects

During the earthquake the plant was shut down for maintenance and no cascading effects in or outside the plant were observed.

2.1.3.4   Contributing factors

Located on a hill top, the tanks were probably subjected to more intense ground motions than other facilities at the site, because of amplification effects of the underlying soil and the higher elevation.

2.2   LESSONS LEARNED FROM EARTHQUAKES

2.2.1   Systems weaknesses and critical components

The above cases suggest that non-anchored storage tanks, and more specifically atmospheric tanks, are particularly vulnerable to earthquake-induced shaking. Major accidents were caused by tank deformation and failure with losses from the tank shell or the roof top due to liquid sloshing, and breaking of flanges and pipe-tank connections. This is in agreement with the findings of several other studies related to earthquake-triggered Natech accidents (e.g. Krausmann et al., 2011, 2010; Steinberg and Cruz, 2004; Salzano et al., 2003). Pressurised tanks generally perform better during earthquakes as their operating conditions require a higher shell

thickness than for atmospheric tanks. This renders them more resistant to earthquake shaking.

Liquid sloshing, in particular for full or nearly full tanks, is an important contributing factor for tank collapse which can lead to the instantaneous release of the complete tank inventory (Ballantyne and Crouse, 1997). The dynamic loading on the tank wall should be considered in the risk assessment in earthquake-prone areas.

On-site piping and pipelines are also susceptible to earthquake shaking.

2.2.2   Potential for propagation

The risk of cascading effects is very high during earthquakes impacting a refinery or other hazardous installations. On the one hand, earthquakes can trigger multiple releases at a single chemical facility or from several affected hazardous installations at the same time, potentially overloading emergency response. On the other hand, the ignition probability is extremely high when flammables are released. Furthermore, in the case of explosions, missile projection could cause secondary accidents in neighbouring installations. Land-use- and emergency-response planning decisions should consider this potential for domino effects.

2.2.3   Consequence severity and extent

Since storage tanks usually hold large quantities of hazardous materials the risk of severe and extended consequences can be high in case of damage. Furthermore, historical analyses showed that several equipment items of the same type can be damaged or destroyed per earthquake event, as well as more than one equipment type. This is compounded by the likely loss of lifelines needed e.g. for cooling processes or for combating fires. On-site emergency planning needs to take this into account and standalone backup plans for accident prevention and mitigation should be developed in earthquake-prone areas. These backup plans should not rely on the availability of off-site emergency-response resources as they might be needed elsewhere to fight the consequences of the earthquake on the population. At the same time, off-site emergency plans should take into account the possible impact of hazardous-materials releases on rescue operations.

Economic losses due to earthquake impact at refineries and petrochemical facilities can be major due to damage to equipment, raw materials and products. Costs related to business interruption and the suspension of production can exacerbate a company's losses.

2.2.4   Protection measures and systems

The risk of earthquake impact on hazardous installations located in earthquake-prone areas can be reduced by performing a risk analysis to ensure that adequate prevention and preparedness measures are in place. This is of particular importance in situations where hazardous-materials releases can occur simultaneously from

different sources over an extended area, and in which on- and off-site utilities might be lost. An assessment of the vulnerability of the emergency resources themselves is also called for.

Seismic building codes based on a realistic assessment of the expected earthquake severity and the resultant seismic loading on structures need to be implemented and compliance monitored. Where not mandatory, it might also be beneficial to extend seismic design codes to cover industrial equipment rather than only buildings. Since also safety barriers, e.g. catch basins or sprinkler systems, may fail under earthquake loading, critical active and passive safety barriers should also be designed to resist the design earthquake (Krausmann et al., 2011). Furthermore, it is crucial that functioning safety-management oversight in companies and effective control mechanisms by authorities are in place to ensure the correct implementation of safety regulations. As the fires at the Cosmo Oil refinery showed, any system or organisational weaknesses due to bad safety culture are amplified by the occurrence of a natural disaster and can result in catastrophe.

Structural protection measures to increase the resistance of the most vulnerable equipment to earthquake shaking exist. Adequate anchoring or restraining of tanks and other types of equipment could avert lateral displacement and/or uplifting and help to keep the equipment intact. Rigid pipe-tank connections which are vulnerable to shaking damage and failure could be replaced by specific flexible connections in earthquake-prone areas.

With warning times ranging from only a few seconds to a couple of minutes, the implementation of early-warning and rapid-response systems at facilities is of only limited use. At most, automatic valve closure and emergency shutdown upon activation of a sensor net could be envisaged although safety-valve isolation requires 10 minutes for equipment at atmospheric pressure and about 3 minutes for pressurised equipment (van den Bosch and Weterings, 1997; Salzano et al., 2009). At Tupras refinery new gas detectors and automatic shut-off systems were implemented following the experiences during the Kocaeli earthquake.

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2.3   TSUNAMIS

2.3.1   Great East Japan earthquake tsunami, Japan, 11 March 2011

2.3.1.1   *Trigger characteristics*

The Great East Japan earthquake caused a tsunami 130 km off the coast of Miyagi Prefecture which inundated over 400 km^2 of land (Mori et al., 2012). The tsunami damage extended over 2,000 km of coastline (Mori et al., 2011). Tsunami heights of 7.3m or higher were observed in Fukushima Prefecture (JMA, 2011), and Mori et al. (2012) report mean inundation heights of 10-15 m north of Sendai. Infrastructure

damage, including rivers, roads, ports, wastewater, and airports, was estimated at around 24 billion US$ (EERI, 2011; Percher, 2014). Reliable historical sources confirm that the area is prone to potentially large tsunamis (Epstein, 2011) with resulting high levels of tsunami preparedness. However, tsunami severities like the one triggered by the Tohoku earthquake were not expected and hence not considered. In this context, the event could be characterised as a perfect storm or possibly even a black swan event.

At the site of the JX Nippon Sendai oil refinery the tsunami inundation height was between 2.5 and 3.5 m. In the western section of the refinery which burned down, the inundation height was about 3 m (Shiogama City Fire department, personal communication). The refinery was also subjected to severe earthquake shaking. The PGA sensors stopped measuring at 0.45 g (the refinery automatically shuts down at 0.25 g) although the actual shaking forces likely exceeded this value. Nonetheless, the earthquake caused only minor spills on some tank roofs due to liquid sloshing and in the processing area of the facility (Krausmann and Cruz, 2013).

2.3.1.2 Impact dynamics

Damage and failure mechanisms

Multiple accidents occurred at the Sendai refinery at the same time. When the tsunami hit, a tanker truck was loading hydrocarbons in the western refinery section. The truck was overturned by the tsunami and a pipe broke near the truck, continuously releasing gasoline that was spread with the tsunami waters and ignited. The tsunami also caused multiple pipeline breaks and many small hydrocarbon leakages from pipe connections. In two places, spills were caused by pipe breaks due to direct tsunami impact. In one case the tsunami caused a heavy oil tank to float. Once the waters receded the tank fell back on the ground, thereby breaking an attached pipe. In both cases significant amounts of heavy oil were released (Sendai City Fire Department, personal communication).

Pipelines were damaged when a ship crashed into one of the refinery's piers due to the tsunami. However, no spills occurred.

Protection measures and systems

With the earthquake epicentre lying 130 km east of Sendai, the refinery on the Sendai City shoreline was directly in the tsunami's path. The tsunami sea walls in the area provided only little protection against the incoming water masses.

The storage tanks were provided with containment dykes to catch accidental releases, e.g. when the tank-pipe connection at one large tank was severed by the tsunami, leading to the release of heavy oil. However, while these dykes serve the purpose of keeping the releases contained, they are not designed to keep the tsunami waters out. The flooding of the containment dykes helped spread the released hydrocarbons.

On-site response to the fires was not functional, as the fire-fighting equipment suffered damage by the tsunami. In addition, external response efforts were significantly delayed, as the refinery access roads were inaccessible due to debris swept up by the tsunami. Overall, fire fighting started only 4 days after the tsunami due to tsunami warnings, the evacuation of personnel, and the blocking of access roads with debris (The Chemical Engineer, 2011; Sendai City Fire Department, personal communication). The fires in the refinery were extinguished after 5 days (Argus 2011).

2.3.1.3 Consequences

Damage to the critical infrastructure and its components

The tsunami triggered a major fire in the western part of Sendai refinery which involved a sulphur, asphalt and gasoline tank (Fig. 2.10 and Fig. 2.11), the hydrocarbon (un)loading facility, numerous pipelines and the rail-tank loading area (Krausmann and Cruz, 2013; Sendai City Fire Department and Shiogama City Fire Department, personal communication). The breaking of pipes connected to tanks caused releases of 4,400 m^3 and 3,900 m^3, respectively, of heavy oil (Fig. 2.12).

The earthquake triggered small hydrocarbon releases on tank roofs, and minor spills from damaged pipes in the processing area.

Socio-economic impact

At the Sendai refinery four people were killed by the tsunami. They were working frantically to untie a crude-oil tanker from the refinery's pier prior to the arrival of the tsunami (JX Nippon Oil and Energy, 2013). In addition, a toxic cloud was formed when a tank containing sulphur ignited, which resulted in an evacuation order in a 2 km radius around the refinery (Krausmann and Cruz, 2013). Trees and grass were covered with thick crude oil from the accident at a nearby park (Daily Caller, 2011).

In terms of economic damage, JX Holdings, Inc. published a loss of 92 billion ¥ (1.2 billion US$) incurred for the fiscal year 2010. The majority of these losses were caused by restoration costs and to a lesser degree by loss on extinguishment of inventory and fixed assets, as well as costs during suspended operations. Significant expenses were expected for the fiscal year 2011 due to fixed costs while the refinery was shut down (JX Holdings, 2011). The Sendai refinery experienced a one-year downtime and full operations were resumed on 9 March, 2012.

Fig. 2.10 Part of the Sendai refinery's western section that was consumed by flames (©2011 Google, ZENRIN)

Fig. 2.11 Burned hydrocarbon tank at the JX Sendai refinery (TCLEE, 2012)

Fig. 2.12 Heavy oil release due to tsunami-triggered damage at the Sendai refinery (©2011 Google, ZENRIN)

Cascading effects

No cascading effects were reported. However, the refinery fires constituted a major risk to a Sendai City Gas LNG (liquefied natural gas) tank that was sandwiched between the fires at JX refinery to its north and tsunami-triggered flammable hydrocarbon releases at another facility to its south. The fire fighters had to keep these releases from igniting as the LNG tank might not have withstood heat impingement from both sides, possibly creating another major accident and resulting in a severe natural-gas shortage in Sendai City (Krausmann and Cruz, 2013).

2.3.1.4   Contributing factors

When the tsunami hit the JX refinery, an oil tank was in the process of being filled. As per normal operating procedures, the valve on the tank-pipe connection was open. Consequently, a significant amount of oil was released upon wave impact when the pipe connection was torn off (Krausmann and Cruz, 2013).

2.4   LESSONS LEARNED FROM TSUNAMIS

2.4.1   Systems weaknesses and critical components

Unanchored tanks and other equipment are vulnerable to floating and displacement due to buoyancy and overturning by the impinging water forces. These phenomena can lead to the breaking of pipe connections, ripping off of valves, and destruction of

equipment, and hence to releases of possibly toxic and/or flammable materials. Debris impact can result in additional damage or destruction.

Refinery port terminals were also identified as vulnerable to tsunami impact, in particular when tankers are docked when the tsunami hits. Tankers engaged in product transfer activities and connected to the (un)loading arm can tear loose upon wave impact, thereby breaking pipe connections or the (un)loading arms which results in hazardous-materials releases. This was observed during the tsunami triggered by the Kocaeli earthquake in 1999 (Steinberg and Cruz, 2004) and by a submarine landslide on Stromboli island in 2002 (Maramai et al., 2005). In this context, the interconnectedness with the power grid needs to be highlighted. During the Tohoku earthquake tankers moored at the terminals in the Kashima industrial park could not disconnect from the (un)loading arms due to the power outage. When the tsunami arrived, the arms broke and the ships floated off, causing damage to berths (Kamisu City Policy and Planning Division, personal communication).

2.4.2 Potential for propagation

In the case of natural disasters with a large impact zone, such as tsunamis, multiple and simultaneous hazardous-materials releases are to be expected. This increases the risk of cascading effects to facilities within the same installation or to other neighbouring installations. In addition, the tsunami waters can disperse flammable spills (including releases from a preceding earthquake) over wide areas, thereby increasing the risk of ignition and severe secondary consequences. This risk is particularly important in situations where failed land-use planning puts industrial facilities in close proximity to residential areas or where separation distances between facilities are insufficient. This potential for cascading effects should be considered in the risk management of hazardous industry situated in tsunami-prone areas.

2.4.3 Consequence severity and extent

Since the tsunami waters would widely disperse any releases of flammable substances, the risk of large-scale fires or many smaller fires distributed over large areas is high. More severe and extended consequences are therefore to be expected. In addition, the potentially wide-scale disruption of lifelines, like for major earthquakes, can hamper effective emergency response, e.g. due to a loss of fire-fighting or cooling water. Loss of lifelines can also significantly decrease a facility's production capacity even if it did not suffer damage by the tsunami.

2.4.4 Protection measures and systems

As a cardinal rule, land-use-planning restrictions should ascertain that industrial development is limited in tsunami-prone areas. However, this is difficult to implement retroactively, in which case supplementary measures are required to protect a hazardous installation.

Offshore break walls or onshore barriers can contribute to reducing the tsunami force. The latter can also help to keep tsunami-driven debris from washing into a facility where it can cause significant damage due to collisions with equipment containing hazardous materials.

In addition, structures or equipment containing hazardous substances, as well as safety-critical systems should be protected from water intrusion or wave-load damage (e.g. storage tanks, fire-fighting equipment). Anchoring of tanks or equipment in general should keep it from floating off its foundations under most conditions. If critical structures cannot be hardened to tsunami impact, relocation out of harm's way (e.g. to higher elevations on the site) should be considered and might be more cost-effective. Sendai refinery has, for example, moved its (un)loading facility northeast on their site to a location less exposed to tsunami impact (Krausmann and Cruz, 2013). Containment dykes around storage tanks could provide some flood protection in case they are not overtopped or eroded, but their primary purpose is to retain accidental releases of hazardous substances rather than keep the flood waters out.

Early-warning systems play a pivotal role in the reduction of natural-disaster risk. Their usefulness for preventing chemical accidents caused by natural disasters is less obvious as the often short warning times would not always allow timely preventive action to be taken. Salzano et al. (2009) discuss the effectiveness of early-warning systems for preventing Natech accidents due to selected natural hazards. With the necessary time for action depending on the type of potentially impacted equipment, the substance it contains, operating or storage conditions, as well as associated actions of people and systems, timely early warning prior to a tsunami would enable operators to take some protective measures. These are e.g. safety valve isolation, plant shutdown, depressurisation of equipment, de-inventorying of equipment and transfer of hazardous substances to safer locations. Tankers moored at a refinery's oil terminal require a warning lead time of several hours to safely stop (un)loading and move into deeper waters (Eskijian, 2006).

Structural protection measures need to be supplemented by organisational measures to reduce the risk of tsunami impact if a refinery is located in a tsunami-prone area. Tsunami hazard management plans should be drawn up at both plant and community level, and construction practises and compliance with building codes need to be monitored. The damaging effects of a possibly preceding earthquake should be considered, as weakened shore-protection systems and industrial facilities have less resistance to an impacting tsunami wave (Cruz et al., 2011). In addition, the vulnerability of emergency resources should be assessed, as they might also be affected by the tsunami. Sendai refinery is now safeguarding its emergency-response equipment on an artificially created hill, from which it will also coordinate response activities in case of a future tsunami (Krausmann and Cruz, 2011).

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3   Large dams

3.1   EARTHQUAKES

3.1.1   Wenchuan earthquake, China, 12 May 2008

3.1.1.1   Trigger characteristics

On 12 May 2008 at 14:28:01 local time, a major earthquake with M_s 8.0 and a focal depth of about 14 km devastated the Wenchuan area in Sichuan Province in the heartland of China (Chinese Earthquake Administration, 2008a). The earthquake occurred along the Longmenshan fault zone and lasted approximately 160s (Zhao and Taucer, 2010). The rupture propagated from Wenchuan north east, rupturing a total of about 300 km with maximum observed displacements reaching 9 m (Chinese Earthquake Administration, 2008b). Zhao and Taucer (2010) report maximum horizontal peak ground acceleration values of 0.98g, and 0.97g in the vertical direction, highlighting the importance of the vertical component in the near-source area.

Wang (2008) indicated that the earthquake appeared to be an entirely unexpected event. However, prior to the earthquake, a study warned that activity on the margin-parallel faults in eastern Tibet could represent a significant seismic hazard to the densely populated Sichuan Basin (Densmore et al., 2007).

The 156 m high Zipingpu dam is an embankment dam of recent construction that stands 17 km from the epicenter along the Min River (Wieland and Chen, 2009). The concrete-face rock-filled dam spans almost 664 m at the crown where its width is 12 m (Zhao and Taucer, 2010). The Zipingpu reservoir has a capacity of 1.12 billion m^3 and an associated power station with an installed capacity of 760 MW (Xinhua, 2002). The dam was designed to withstand a seismic intensity of 8 on the 12-degree Chinese seismic intensity scale with a design peak ground acceleration of 0.26g (Wieland and Chen, 2009), while the intensity in the epicentral area reached XI (Wang, 2008). At the time of the earthquake the reservoir was filled less one third of its capacity (China Dialogue, 2008).

3.1.1.2   Impact dynamics

Damage and failure mechanisms

On-site field surveys of the dam after the Wenchuan earthquake found a number of failures (Lekkas, 2008):

- Subsidence of the crown in the central part of the dam,
- Deformation of the lower dam face,

- Deviations and deformations of the construction elements throughout the dam face,

- Widening of construction joints,

- Extended massive landslides into the reservoir,

- Landslides and rock falls on both left and right abutments, causing additional damage to secondary constructions.

After the earthquake, a maximum settlement of over 68 cm was measured at the upper middle of the concrete wave protection of the dam. This value increased to more than 74 cm five days later and then remained stable. Inside the rock-fill dam, measurements indicated that 24 m below the crown the settlement reached 81 cm (Zhao and Taucer, 2010). The dam underwent permanent horizontal deformations of the embankment in the downstream direction of almost 20 cm at the top of the wave protection. The maximum estimated horizontal deformation at the crown was 60 cm (Chen et al., 2008). Fig. 3.1 shows an example of damage due to the earthquake.

The machine room that raised the sluice gates also suffered damage. Consequently, the gates could at first not be opened to release water. The gates were forced open five days after the earthquake (China dialogue, 2008). Some walls of the power plant and other buildings collapsed, and some partly sunk.

Protection measures and systems

The Zipingpu dam was designed to withstand earthquake intensity levels of 8 on the Liedu scale (Wang, 2008). With the intensity close to the epicenter significantly exceeding this level, damage to the dam was inevitable.

The dam had an emergency spillway that allowed the partial discharge of the reservoir following the earthquake to release pressure on the dam and minimise the risk of dam failure (Lekkas, 2008).

3.1.1.3 Consequences

Damage to the critical infrastructure and its components

The Wenchuan earthquake caused damage to the dam body, as well as to safety-critical elements and appurtenant structures. Damage to the machine room resulted in the inability to raise the flood gates which must be operational after a strong earthquake. Power generation buildings and equipment were also damaged and had to be shut down (Wieland and Chen, 2009; China Dialogue, 2008).

At the time of the earthquake, the water level in the Zipingpu reservoir was only about one third of its nominal capacity. It is unclear how the dam, the concrete face, and the waterproofing system would have performed had the reservoir been full (Wieland and Chen, 2009).

Socio-economic impact

The earthquake killed almost 70,000 people, injured over 374,000 and left 5 million homeless. There are estimates that over 5 million buildings collapsed, while 21 million buildings were damage in the earthquake (USGS, 2008). The economic losses due to the earthquake amount to over 960 billion RMB or about 140 billion US$ (Shi, 2008). The economic losses due to infrastructure damage and destruction are estimated as 185 billion RMB or 27.5 billion US$ (China State Council, 2008a).

The area impacted by the Wenchuan earthquake is large and rich in reservoirs that were built over the last 50 years for the generation of electric power, to provide water for irrigation, and to support flood control. The China State Council (2008b) indicated that overall 2,473 reservoirs were affected by the earthquake. Two weeks after the earthquake, 69 reservoir dams were on the verge of failure, requiring emergency drainage to prevent dam collapse (RMS, 2008). The condition of another 310 dams was considered to be "highly dangerous".

Fig. 3.1 Fracture at the Zipingpu dam's crown (taken from Lekkas, 2008)

Cascading effects

No cascading effects occurred as the integrity of the dam was not compromised by the earthquake and the reservoir could be partially discharged. This was important in view of the heavy rains following the earthquake. Had the dam failed, the consequences to downstream towns would have been catastrophic.

There was a risk that dams upstream of the Zipingpu dam and damaged by the earthquake would fail in case of strong aftershocks. However, industry and government experts maintained that the Zipingpu dam could have held back the extra water (China Dialogue, 2008).

The cliffs around the reservoir were in danger of collapsing and many major landslides entered the reservoir. The reservoir was not filled to capacity and no overtopping occurred.

3.1.1.4 Contributing factors

There is controversy as to whether the Zipingpu reservoir might have contributed to triggering the Wenchuan earthquake tens or hundreds of years in advance due to the stress change induced by the large water body of the reservoir on the rupturing fault (Chen, 2009; Deng et al., 2010; Ge et al., 2009; Kerr and Stone, 2009, 2010; Zhou et al., 2010).

3.2   LESSONS LEARNED FROM EARTHQUAKES

3.2.1   Systems weaknesses and critical components

Large hydropower plants consist of different components that are more or less safety relevant. The dam body's structural integrity needs to be ensured after an earthquake, as well as the operability of certain elements that are considered safety critical. These are, for instance, bottom outlets, spillways and related hydro- and electromechanical equipment (Wieland, 2012). Damage to these components can hamper the capability to release pressure on the dam by discharging water from the reservoir and therefore seriously endanger the dam's integrity. Earthquake impact on appurtenant structures, such as powerhouses, switchyards, etc. do not pose a threat to the dam itself but damage will lead to service disruption and economic losses.

Earthquakes pose a risk to large dam projects not only due to ground shaking but also because of potential fault movements, landslides, rockfalls, and liquefaction etc. (Wieland, 2012). The Wenchuan earthquake demonstrated the danger of these additional hazards, and in particular the risk of mass movement to hydropower plants in steep mountain valleys. What aggravated the recovery situation was the blocking of access roads by rockfalls, making it impossible for construction equipment to reach several dam sites for several months (Wieland, 2012). This means that in an emergency situation, damaged dams might have to remain safe for an extended time span before rehabilitation can begin.

3.2.2   Potential for propagation

The risk of cascading effects can be significant when dams are impacted by strong multiple earthquake shocks. On the one hand, failure of the dam endangers the downstream environment, potentially causing a high number of fatalities. On the

other hand, strong earthquakes can affect a large area and many dams may be subjected to strong ground shaking at the same time. The earthquake-induced failure of upstream dams and the subsequent water masses rushing downstream might exceed a downstream dam's capacity for dealing with the extra water load, potentially causing it to fail under the additional pressure.

In mountainous areas there is a risk of earthquake-triggered large landslides that enter a dam's reservoir. Depending on the amount of mass involved in the landslide and the water level in the reservoir, there can be a significant risk of overtopping.

3.2.3   Consequence severity and extent

Large dams are high-risk infrastructures due to the hazards involved in the damming of billions of litres of water. Therefore, if a dam fails, consequences can be severe and extended. While the overall performance of large dams under strong earthquake loading is encouraging, the Wenchuan earthquake highlighted several weaknesses and gaps that need to be addressed in future reviews of dam safety standards.

3.2.4   Protection measures and systems

Seismic dam design exists since the 1930s and large dams generally have a good seismic safety record (ICOLD, 2010a). There is only a single case where lives were lost in a dam failure triggered by the Tohoku earthquake in 2011 (Matsumoto et al., 2011). Until now, only embankment dams have suffered complete failure due to seismic loading while large concrete dams exposed to strong earthquakes were damaged but did not fail (Wieland and Brenner, 2008).

Updated seismic design guidelines and recommendations are available that should ensure the safe performance of large dams during earthquakes (ICOLD, 2001, 2002, 2010b). These guidelines address the design features of dams and appurtenances to effectively resist seismic ground motion, and they support the selection of seismic parameters for large dams. These guidelines render the previously applied pseudostatic analyses for seismic dam design obsolete and instead introduce modern seismic design criteria and propose dynamic analyses to calculate the inelastic seismic response of embankment and concrete dams (Wieland, 2012). More specifically, ICOLD (2010b), which is a revision of the ICOLD Bulletin 72 published in 1972 and which guides the selection of seismic parameters, suggests different design criteria for the various structures and elements of a large dam (Wieland, 2012). Different levels of design earthquake severities are considered as a function of safety criticality of the dam component.

The Wenchuan earthquake highlighted the multi-hazard nature of earthquakes and its potential for causing secondary hazards. However, the focus in the seismic design of large dams is usually on the resistance to ground shaking and does not address the potential multi-hazard nature of strong earthquakes (Wieland, 2012). However, these secondary hazards may have worse consequences than the triggering earthquake and a comprehensive hazard assessment assuming a full

reservoir should be carried out that includes all possible hazards. Wieland (2012) suggests to prepare a hazard matrix that captures all hazards related to the earthquake for each component in a given dam project and assign specific design actions.

There is concern related to existing large dams whose construction dates back to a time when design criteria and analysis methodologies differed significantly from what is considered adequate nowadays (ICOLD, 2010a). Compliance of these dams with today's safety criteria should be checked. Where unacceptable performance is predicted, retrofitting measures can improve the situation. For embankment dams in situ soil improvement, removal and replacement of weak soils, embankment buttressing and combinations of these methods have been proposed (ASDSO, 2014). Concrete dams can be retrofitted, e.g. through anchoring and buttressing.

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3.3 FLOODS

3.3.1 Central Europe floods, Poland, 7 August 2010

3.3.1.1 *Trigger characteristics*

A series of devastating weather events swept over Central Europe in spring and summer 2010. Poland was the worst affected country with widespread property damage and some two dozen fatalities (Reuters, 2010a). The catchment area of the Neisse River, a border river between Poland and Germany, was particularly affected by heavy rains on 7 and 8 August. The rain triggered extensive flooding of a magnitude that last occurred in the Neisse River catchment a century ago. At some gauge stations the 100-yr flood levels were exceeded (Jelonek et al., 2010).

The Niedow dam is an earthfill embankment dam of 18 m height and 270 m length that is located on the Witka River in Poland a short distance before it enters the Neisse River (Fry et al., 2012;). The dam was constructed in 1962 and its reservoir, which has a storage capacity of 4.8 million m³, provides water to several power plants. The maximum discharge via the dam's spillways is 500 m³/s (Jelonek et al., 2010).

Following the heavy rains, the Niedow dam failed at about 18:00 on 7 August 2010. About two hours before, over 500 m³/s discharge from the reservoir were measured. Using data from the Reczyn gauge downstream of the Niedow dam, a water level of 572 cm was estimated before dam failure.

3.3.1.2 *Impact dynamics*

Damage and failure mechanisms

The inflow of water from the Witka River's catchment area into the Niedow reservoir exceeded the discharge capacity of the dam via the spillways which was limited to 500 m³/s. With no flood storage capacity available, the dam crest was overtopped which led to the washout of the embankment and consequently to the failure of the earth-fill dam (Jelonek et al., 2010). The destroyed dam is shown in Fig. 3.2.

Protection measures and systems

The Niedow dam was equipped with spillways whose discharge capacity was, however, not sufficient to cope with the water masses that were accumulating in the reservoir.

3.3.1.3 *Consequences*

Damage to the critical infrastructure and its components

The embankment dam was completely destroyed by the flood.

Fig. 3.2 Failure of the Niedow dam in Poland due to flooding (taken from Fry et al., 2012)

Socio-economic impact

The Neisse River floods of August 2010 caused the worst flooding since the 2002 summer floods that affected Central Europe. The failure of the Niedow dam aggravated the situation for some areas already in distress, including Görlitz and Zittau in Germany and Bogatynia in Poland. Bogatynia was completely inundated in less than an hour and did not receive any advance warning. At least 2,000 flood victims required evacuation to higher-lying areas. It is believed that the release of the Niedow reservoir due to the dam collapse increased the speed of the water surge (Reuters, 2010b).

Cascading effects

The failure of the Niedow dam contributed to the dramatic increase of the Neisse River's water level in Görlitz county, triggering a disaster alert in an area that was already heavily affected by the floods (Epoch Times, 2010). The Neisse level increased within only a few hours to over 7 m, leading to the evacuation of about 1,000 people. In contrast, the yearly average water level of the Neisse River is 1.7 m (Sächsische Zeitung, 2010).

3.3.1.4 *Contributing factors*

In some areas, increased precipitation levels in July and rain at the beginning of August had left the soil saturated with water. Consequently, the ground's capacity to absorb the rain water was diminished, leading to increased surface runoff. This

significantly influenced the hydrological reaction to the heavy rains (Jelonek et al., 2010).

3.4   LESSONS LEARNED FROM FLOODS

3.4.1   Systems weaknesses and critical components

Floods are an important accident initiator in large dams with respect to both failure frequency and severity (Charles et al., 2011). Regardless of construction type, dams are at risk when the water inflow into the reservoir greatly exceeds the outflow capacity because of inadequate spillway design or where there is no emergency spillway (Evans et al., 2000). This can lead to failure of the dam by overtopping and scouring of the embankment or foundation. In the US, about 30% of dam failures over the last 75 years were caused by overtopping of embankment dams (USDOI, 2012).

Where spillways are gated, there is the additional risk of gateway malfunction due to mechanical failures, loss of power or gate binding. Gates can also be blocked by floating debris, vegetation, ice, or landslides and rockfalls from adjacent unstable slopes (Fell et al., 2000; USDOI, 2012).

Embankment dams are vulnerable to even low levels of overtopping, if sustained, while concrete dams would likely be able to withstand a certain level of overtopping due to their rock foundations (USDOI, 2012). In Switzerland, for example, overtopping may be accepted for a Probable Maximum Flood (PMF) at concrete dams as long as the water level remains below a so-called safety level, which is typically 0.5 to 1.0 m above the concrete dam crest (Schleiss, private communication). The depth and duration of the overtopping is a key factor in determining the risk to a dam, as well as the erodibility of the embankment material or rock foundations.

3.4.2   Potential for propagation

Considering that large rivers usually have a network of multiple dams, there can be a risk of cascading failure triggered by the collapse of an upstream dam. This can be of concern during flood conditions when downstream reservoirs might already have reached their flood retention capacity and where they consequently might not be able to accommodate the water volume approaching from the reservoir of a failed upstream dam. Safe dam design in addition to considering the need for excess storage capacity for flood conditions ensures that this risk remains low.

Another cascading scenario, although extremely low probability, exists in situations where multiple dam failures occur within a short time interval in the same river basin affected by floods. Probably the most tragic event occurred in August 1975 in the Huai River basin in Henan Province. After a period of intense rain caused by Typhoon Nina, the Banqiao dam on the Ru River failed after overtopping, causing

severe flooding in the downstream towns and villages (People's Daily Online, 2005). The situation was aggravated by the almost contemporary failure of other dams in the area. Flood diversion areas were evacuated and inundated, and several dams were intentionally destroyed in order to control the release of water in selected directions. Overall, the floods resulted in the destruction of 62 dams, either by failure due to overtopping or air bombardments to protect other dams, unleashing about 6 billion m³ of water to an area of about 10,000 km^2 (People's Daily Online, 2005). According to official statistics released only 30 years after the event, 26,000 people perished in the floods. A number of factors were blamed for the catastrophe, among which the inability to predict the heavy rain based on the then current scientific knowledge, insufficient dam design resulting in a lack of spillway capacity, and the absence of an alert system or evacuation plan for the population.

Heavy rains not only cause floods but they can also trigger mass movements in mountain areas with unstable slopes. Large landslides entering the reservoir can lead to dam overtopping with potentially severe consequences for the downstream environment.

3.4.3   Consequence severity and extent

Dam failure during flood conditions releases the full reservoir volume to the downstream environment over a relatively short period of time. The discharged amount of water also includes the flood volume stored in the reservoir up until the time of collapse. The consequences to downstream settlements can therefore be both disastrous and spatially extended. Sensible land-use planning should therefore restrict settlements in high-hazard areas, and early-warning systems linked with evacuation plans will reduce the risk for existing towns and villages.

3.4.4   Protection measures and systems

Dam safety during flood conditions is ensured by designing the dam to withstand the Probable Maximum Flood. This is a flood volume which can be spilled (and partly stored) without endangering dam stability (Lempérière and Vigny, 2005). If a dam's spillways are adequate to safely handle the PMF, then the loss of storage capacity should not be able to impact the dam structure. This holds for recently constructed dams which usually also include emergency spillways as an added protection mechanism in high flood-risk areas (Evans et al., 2000).

Definition of the PMF requires a good knowledge of the hydrological situation at the dam site which can, however, change over time. Lempérière (2006) indicates that global warming may increase by 20% on average the flood volumes and peaks, critically lowering the return period of severe and potentially extreme flood events. Furthermore, time series for flood statistics used for spillway design may have been too short or not representative. As a consequence, many older dams may have been designed for floods that no longer represent the magnitude of the PMF for a given site and which may be at an increased risk of overtopping and failure during floods

(USDOI, 2012). It is therefore important to update the PMF where it is not current, and to link it with retrofitting measures, such as upgrading of spillways, where necessary and possible. Other recent technological improvements include wave and erosion protection systems and wave walls (Charles et al., 2011). In cases where retrofitting is not feasible either due to technological challenges or because costs are prohibitive, the reservoirs would have to be operated at a lower level to safely manage PMF inflow by retention. In a further step, the relocation of settlements or other assets downstream of the dam might be considered.

In the case of gated spillways, in many countries the rule n-1 is applied for the design flood, i.e. the design flood has to be released even if one the spillway gates, namely the one with the largest capacity, would not be operational. Regarding floating debris, a minimum width of the spillway passages of 10 m is recommended in alpine regions to avoid clogging by floating trees (Pougatsch et al., 2011).

Lempérière and Vigny (2005) question the traditional approach to base the overall dam design on a design flood of about 1,000 years return period for which the reservoir level is kept well below the dam crest. Instead, they propose the use of a safety check flood of very low probability (often chosen as the PMF), which is also advocated in ICOLD Bulletins 82 and 125 (ICOLD, 1992, 2003). The safety check flood is the most extreme flood that the dam can withstand without failure but also with a low safety margin. Limited overtopping may be permitted for concrete dams; for embankment dams overtopping is not allowed (ICOLD, 2003). Some limited damage to waterways or loss of fuse devices is acceptable (Lempérière and Vigny, 2005). The use of the safety check flood provides a more realistic approach to dam safety design and is more cost effective. Recently, ICOLD has issued Bulletin 142 on the safe passage of extreme floods that addresses the confidence level of design flood estimates and strategies for planning spillways for floods exceeding the design floods (ICOLD, 2012). Switzerland and many other countries have implemented for a long time the above-mentioned dual concept of design flood and safety check flood (Schleiss and Pougatsch, 2011).

Methodologies for estimating the failure probability of embankment or concrete dams with respect to different triggering mechanisms are in varying stages of development. Fell et al. (2000) indicate that flood-induced overtopping of embankment dams and liquefaction failure modes lend themselves to analysis in a Quantitative Risk Assessment context. However, the estimation of the failure probability of concrete dams due to overtopping and foundation scouring requires expert judgment which introduces additional uncertainty in the safety analysis.

Safe dam design is the most important factor for controlling risks from large dams. Additional factors are maintenance, regular inspection and repair, as well as the preparation of emergency plans downstream of the dam. With sometimes substantial warning times, the implementation of early-warning systems coupled with evacuation plans, can contribute substantially to reducing the risks to the downstream environment.

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4 Hydrocarbon pipelines

4.1 EARTHQUAKES

4.1.1 Northridge Earthquake, USA, 17 January 1994

During the Northridge earthquake a wide range of damage to critical infrastructures was observed both due to seismic shaking and ground failure phenomena. This case study discusses the rupture of an oil pipeline during the earthquake and the subsequent oil spill into a river. Further overview information on the general earthquake effects is provided in Section 2.1.3.

4.1.1.1 Trigger characteristics

The Northridge earthquake occurred at 4:30 a.m. on 17 January 1994. The pipeline rupture and oil spill were discovered at 5:35 a.m. by the ARCO-Four corners company operating the pipeline.

In the area estimated to have the highest ground motion, a peak ground velocity (PGV), the earthquake intensity measure commonly used for pipeline-damage assessment, of 170 cm/s was recorded. This was the highest ground velocity measured to that date (Schiff, 1997). No detailed PGV data over the length of the pipeline was available. However, a seismic station at a filter plant close to one of the pipeline breaks measured 84 cm/s (USGS, 2014).

4.1.1.2 Impact dynamics

Damage and failure mechanisms

A 70-year old 10-inch crude-oil pipeline suffered fracturing at eight different locations due to the earthquake forces. In addition, during repair and testing about a dozen damaged welds were detected over a 50 km stretch (Schiff, 1997). More specifically, the pipeline's acetylene welds, commonly found in old pipelines built in the 1920s and 1930s, failed at all rupture sites. There were indications of girth weld failure and lack of penetration at these locations (Leveille et al., 1995; PHMSA, 2014).

Two petroleum pipelines traversing the Northridge epicentral area but built after 1950 with improved welding techniques did not suffer any damage (Schiff, 1997).

Protection measures and systems

Due to its old age the pipeline had not benefited from noteworthy earthquake-resistant design.

4.1.1.3 Consequences

Damage to the critical infrastructure and its components

Multiple breaks occurred at the welded joints of an old crude-oil pipeline. Numerous welds were damaged but did not fracture.

Socio-economic impact

The pipeline ruptured at different locations along a 50 km stretch. The most important fracture at a booster pump substation led to the emptying of 8 km of pipeline and one connected storage tank onto the ground and into a storm drain, ultimately discharging about 550 m^3 crude oil into the Santa Clara River (Leveille et al., 1995; Schiff, 1997). Another major spill of 160 m^3 also flowed into gutters and storm drains and from there into the Los Angeles River. This crude oil caught fire, resulting in one injury, and damage to cars and houses (Schiff, 1997). It was estimated that overall about 4,600 barrels (ca. 730 m^3) of crude oil were spilled from the ruptured pipeline.

The Santa Clara River contains significant water and wetland habitat areas and is home to two endangered species. The spill impacted fish, birds, mammals and other riparian and aquatic animals and destroyed about 100 acres of riparian vegetation (DFW, 2013). Three years after the oil spill, ARCO settled with the California state government and paid 7.1 million $US in restoration costs. In addition, the company spent "many tens of millions" immediately after the earthquake on clean-up costs, including removing oiled vegetation, excavating soil and sediment, backfilling and grading of the river bed (Burnaby Pipeline Watch, 2014).

Cascading effects

One major spill ignited, causing an injury and damage to property.

4.1.1.4 Contributing factors

The vulnerability of acetylene welds to the earthquake contributed to the oil spill. After the earthquake the state of California launched a survey to assess the dimension of potential earthquake-related problems in acetylene-welded pipelines.

The oil-spill response was initially hampered by other earthquake-related problems, such as e.g. closed access roads, power outages, disruption of the communications infrastructure, and competition for scarce resources (Leveille et al., 1995). This is a common problem during chemical accidents or oil spills caused by natural disasters.

4.1.2 Kocaeli Earthquake, Turkey, 17 August 1999

Due to the proximity of the earthquake to the Gulf of Izmit, a large number of waterfront structures, utilities and tanks were damaged. However, no damage to hydrocarbon transmission pipelines was reported. Therefore, this section also

includes damage to water transmission pipelines, as lessons applicable to hydrocarbon pipelines can be learned.

4.1.2.1   Trigger characteristics

A detailed overview of the characteristics of the Kocaeli earthquake is provided in Section 2.1.2.

4.1.2.2   Impact dynamics

Damage and failure mechanisms

Buried pipes in the Kocaeli earthquake region consist of continuous steel pipes with welded joints and segmented reinforced concrete (R/C) or asbestos cement (A/C) pipes with rubber gasket joints. Continuous pipes usually behave in a ductile manner, whereas segmented pipes are considered to be brittle. The fragility expressions of buried pipes depend on the diameter (large or small) and the pipe (segmented or continuous). The fragility of continuous pipes is generally much lower than that of the segmented pipes (ALA, 2005). Failure of continuous pipelines is likely if they cross active faults or are located in regions susceptible to permanent ground displacements (Erdik, 2000).

Hydrocarbon transmission pipelines are continuous pipes. The natural gas transmission system in the earthquake area consists of steel pipes with welded joints. The Petroleum Pipeline Corporation (BOTAS) which covers all oil and gas imports and distribution pipelines reported no damage on any of their installations. The high-pressure natural gas pipeline that connects Russia with Turkey crosses Izmit Bay about 30 km west of Izmit and the natural gas pipeline connections to industry in the area were operational after the earthquake. Overall, good practices for natural-gas pipeline siting, design and construction seem to have rendered the transmission pipelines less susceptible to the strong shaking levels of the Kocaeli earthquake.

At the TUPRAS refinery an above ground crude-oil unloading pipeline fell from its support structure (Fig. 4.1) and a 71 cm (28 in) pipeline supplying water to the refinery was damaged. Some embedded pipes along the seashore at the Petkim petrochemical facility (SEKA) also suffered damage. The failure of the water supply eventually caused problems in controlling the fires at the Tupras refinery.

There was some damage to major continuous steel water transmission lines, especially where they crossed the fault zone or where severe permanent ground displacements occurred near Arifiye (Erdik and Uckan, 2013; Takada et al., 2001; EERI, 1999). Damage and failure mechanisms included wrinkling and buckling (Fig. 4.2).

Fig. 4.1 The ground-supported unseated crude oil unloading pipeline fell from the supports at TUPRAS refinery due to inertial effects (Courtesy of TUPRAS (2000))
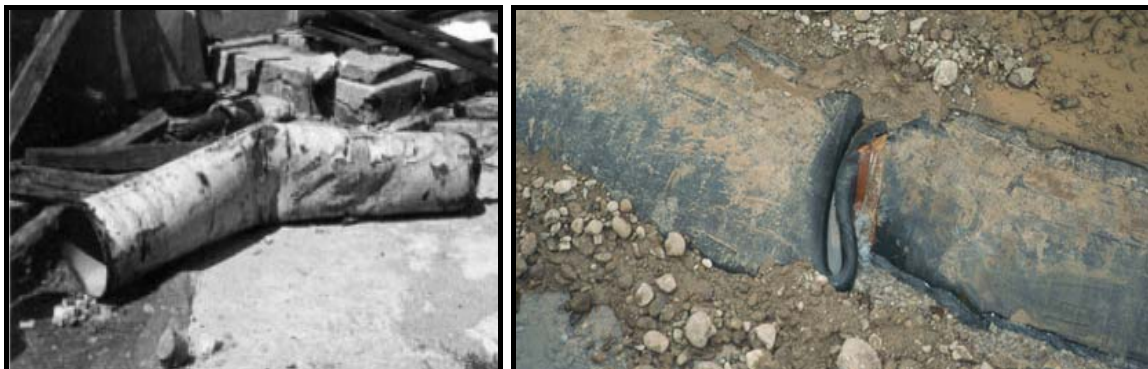
Fig. 4.2 Failure (wrinkling) of a large diameter welded steel pipe crossing the ruptured fault in Arifiye (East of Izmit (left) (Takada et al., 2001) and buckled steel pipe connection (Bilham et al. 2003)

Protection measures and systems

Design safety is an important protection mechanism that paid off during the Kocaeli earthquake. Other protection systems include safety valves to control and close off the fluid flow in transmission and distribution lines. In the water network the valves were operated manually.

4.1.2.3 Consequences

Damage to the critical infrastructure and its components

No damage to oil and gas transmission pipelines was reported during the Kocaeli earthquake.

The water transmission network suffered heavy damage due to earthquake loading. Water treatment plants and pump stations were also vulnerable to earthquake impact.

Socio-economic impact

No damage to the oil and gas transmission network was reported. In addition, there is no quantitative information on the socio-economic repercussions due to damage to the water transmission and gas distribution networks. However, the overall losses due to lifeline damage in the earthquake region are estimated to be of the order of 1 billion US$ (Durukal and Erdik, 2008).

Cascading effects

Since no flammable oil or gas leaks from major pipelines were observed, the risk of ignition and spreading of fires was low.

4.1.2.4 Contributing factors

No contributing factors are known.

4.2 LESSONS LEARNED FROM EARTHQUAKES

4.2.1 Systems weaknesses and critical components

Incident data suggests that old pipelines made of non-ductile materials with oxy-acetylene welds are the system component most vulnerable to ground motion. In this case, weld fractures are the most common failure mode. With more of these pipelines going out of service, the risk of fracturing or damage in earthquake-prone areas is decreasing. Replacement pipelines using newer materials, joint types and welding techniques show a much higher resistance to earthquake shaking. However, also newer pipelines are vulnerable to soil liquefaction with lateral spreading and fault movement.

Buried continuous pipelines generally perform better than above-ground (ground supported) steel pipes due to inertial effects and the potential for unseating from their supports. Furthermore, above-ground pipelines and exposed sections of buried pipelines can be subject to earthquake-triggered landslides in mountainous areas.

While newer pipelines are less susceptible to ground-motion effects, discontinuities like supports and valves and connections to pump stations are vulnerable (Griesser et al., 2004).

Tanks at liquid-fuel distribution terminals that are supplied by transmission pipeline can also be affected by the earthquake. Most common damage is due to tank-pipe breaking or the movement of unanchored tanks.

4.2.2 Potential for propagation

Since hydrocarbon pipelines transport flammable materials, the risk of ignition is high. Therefore, there is the danger of accident propagation if the affected pipeline is traversing residential areas. Furthermore, the earthquake can damage one pipeline in several places, or impact many pipelines at the same time. This increases the likelihood of cascading effects.

4.2.3 Consequence severity and extent

Spills from liquid-fuel pipelines are usually more problematic in terms of spatial extent and impact on people and the environment (Girgin and Krausmann, 2014). Crude-oil and petroleum-product releases on the ground can flow into storm drains that empty into rivers, potentially causing major environmental damage. In addition, flammable spills can ignite and spread over wide areas, thereby posing a risk to the population and property. Clean-up operations for liquid-fuel releases can be extremely complex and costly.

Failure of the water supply by an earthquake or blockage of access roads to spill areas can significantly hamper or delay emergency-response operations, potentially aggravating the consequences of the pipeline failure.

4.2.4 Protection measures and systems

One of the most effective and economical ways to protect pipelines and associated facilities is adequate siting to keep vulnerable equipment out of harm's way. While it may not always be possible to completely avoid earthquake-prone areas, the careful selection of pipeline routes, pipeline orientation with respect to fault lines, and sensible choices for siting critical components, will greatly contribute to reducing the risk of accidents (Yokel and Mathey, 1992).

Design safety is the most important pipeline protection mechanism that relies on the implementation of modern design standards, including the use of more resistant pipe materials and novel techniques for strengthening joints against earthquake shaking. Additional measures are required for pipelines in liquefaction-induced permanent ground deformation zones or at fault crossings. It is common practice to adjust the orientation of the pipeline with respect to the fault or to use low-density backfill material at the trench.

An excellent example of the success of engineering solutions to protect pipelines from seismic activity is the Trans-Alaska oil pipeline. It was built in the 1970's according to stringent earthquake design specifications to accommodate the possibility of a magnitude 8 earthquake from the Denali Fault which the pipeline crosses. This fault ruptured in November 2002 during a magnitude 7.9 earthquake and strong shaking damaged a few of the pipeline's supports near the fault, but the pipeline did not break (USGS, 2003). In view of the possibility of strong earthquakes the pipeline had been arranged in the vicinity of the fault in a zigzag configuration supported on Teflon shoes that can slide on long horizontal beams, allowing the pipe to move back and forth under stress (Fig. 4.3). This measure easily accommodated the 4.3 m horizontal and 0.8 m vertical shift at the fault crossing. The overall cost of implementing this measure was about 3 million US$ (in 1970 US$) which was considered well below potential losses due to lost revenue and repair costs, as well as environmental cleanup, had the pipeline ruptured.

The installation of strong-motion detectors on the pipelines in seismic areas can be an additional measure to support quick operator action in case of an earthquake. Based on the information from the detectors, control signals, such as reducing flow in the pipeline or shutting it down completely, can be issued. This reduces the stresses on the pipeline wall (Griesser et al., 2004).

Fig. 4.3 Trans-Alaska pipeline crossing the Denali fault line (USGS, 2003)

Liquid-fuel distribution terminals that are supplied by transmission pipeline can also be affected by the earthquake. Protection measures for these facilities, including storage tanks, piping and other equipment, are addressed in Section 2.2.

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5 Gas storage and distribution

5.1 EARTHQUAKES

5.1.1 L'Aquila Earthquake, Italy, 6 April 2009

Damage to gas networks were often observed during earthquakes, with different degrees of severity. A significant and well-documented case is the impact of the recent L'Aquila earthquake in Italy. In this case, the response of the gas network was reasonably satisfactory, despite the observed damage to some metering stations, pipelines and valves. The description of the case history of the L'Aquila (2009) earthquake is based on the work of Esposito et al. (2013), who analysed the response of the gas network during the earthquake and the recovery process.

5.1.1.1 Trigger characteristics

On 6 April 2009, at 03:32:40 UTC, a M_w = 6.3 earthquake struck the Abruzzo region in central Italy. The earthquake occurred at about 10 km depth along the Paganica fault, a normal fault located below the city of L'Aquila (INGV, 2009). Cirella et al. (2009) report a rupture duration of 6.5 seconds with a maximum slip of about 1 m.

Considerable damage to structures and infrastructures was detected over a broad area of approximately 600 square kilometers, including the downtown of L'Aquila and several villages in the Aterno river valley. The PGA recorded in the near-source region ranged from 0.33 to 0.65 g, the latter representing one of the highest PGA values measured in Italy (Chioccarelli et al. 2009).

With respect to geotechnical effects induced by the earthquake, evidence of surface rupture was found along the Paganica fault. In particular, in the areas of the Tempera and Paganica villages, a set of well-aligned ground ruptures was observed (Blumetti et al. 2009, Vittori et al. 2011) that caused significant damage to infrastructures sitting on the fault (Dolce et al. 2009). Secondary effects of ground deformation and failure, mainly related to slope instability, collapse of some underground cavities, and ground settlement induced by liquefaction, were reported soon after the earthquake (Monaco et al. 2011). However, geotechnical effects in the area interested by the gas network were minor with respect to reported damage.

5.1.1.2 Impact dynamics

In Italy, the ENEL gas transmission and distribution systems include the following principal components:

1.   high-pressure (HP) transmission lines (at a national scale);

2.   metering/pressure reduction stations (M/R stations);

3.   medium-pressure (MP) distribution networks (at regional scale);

4.   reduction groups (RGs);

5.   low-pressure (LP) distribution networks (at urban scale);

6.   demand nodes (IDU) consisting of buried and above-ground pipes and accessory elements;

7.   gas meters (at the customer scale).

The gas is distributed via a 621 km pipeline network, 234 km of which operating at medium pressure (2.5 – 3 bar), and the remaining 387 km at low pressure (0.025 bar – 0.035 bar). Pipelines of medium and low pressure distribution networks are either made of steel or high-density polyethylene (HDPE). The latter pipes have nominal diameters ranging from 32 to 400 mm, whereas the diameter of steel pipes is usually between 25 mm and 300 mm.

Before 1990, gas welded joints were used for steel pipes. Since 1990, gas welded joints are only used for pipe diameters of less than 250 mm and arc welded joints otherwise. HDPE pipes use fusion joints. The analyzed network was constructed between 1968 and 2009 and the burial depth was usually between 0.6 m to 0.9 m before 1992, and equal to 1 m after 1992.

Damage and failure mechanisms

In the LP distribution network repairs and replacement of steel pipes were made necessary mainly because of breaks or leaks at gas welded joints (Fig. 5.1). Overall, 176 repair/maintenance operations addressed the damage to different buried pipeline network components in the following proportion: pipelines (37%), valves (18%), demand nodes (32%), and mixed (13%). The latter refers to repairs made to more than one component (e.g., repair to pipe and unburied node; or repair to valve and unburied node). Fig. 5.2 illustrates the repair/maintenance operations with respect to the pressure level of the distribution network MP or LP, and pipe material (steel or HDPE). The largest proportion of repairs (72%), and therefore of the damage, was localized in the LP distribution network, made of steel pipes.

Post-earthquake activities also addressed the repair and upgrade (inclusion of safe-stop systems) of the input/output substation network of two M/R stations (Fig. 5.3a). The three M/R stations of the L'Aquila distribution system are cased in one-story reinforced concrete structures with steel roofs and no damage was observed to the buildings. The regulator and mechanical equipment for the M/R stations resulted undamaged. One of the principal causes of earthquake-induced damage to different types of RG stations was the collapse of rubble from adjacent buildings (Fig. 5.3b). As a consequence, some RGs in L`Aquila and Onna had to be replaced concurrently with the laying of new pipe.

Fig. 5.1 Damage to gas pipes following the L`Aquila earthquake: gas welded joint of a LP steel pipe pulled apart in Paganica (Esposito et al., 2013)
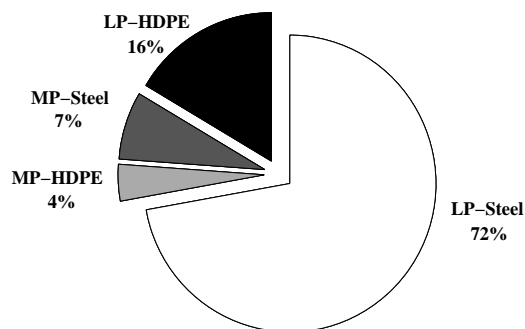
Fig. 5.2 Repair operations addressing damage to buried components for the entire gas network: number of repairs distinguished with respect to pressure level and pipe material (Esposito et al., 2013)

Fig. 5.3 Damage to stations: repairs to the input/output network of Onna M/R and inclusion of stop-system (left); RG housed in a masonry kiosk closed to building and damaged following the earthquake (right) (Photos courtesy of ENEL Rete Gas)

Protection measures and systems

All components contained in both the L'Aquila M/R stations and RGs were unrestrained and therefore sensitive to seismic (inertial) forces. The material and joint type used for the pipelines were designed for recover the strength at joint position and to allow ductile deformations during the motion.

5.1.1.3 Consequences

Damage to the critical infrastructure and its components

The steel pipes in the LP gas network were the most vulnerable to the earthquake due to their gas welded joints. One pipe of the MP distribution network, connected to a damaged bridge in Onna, was replaced with a new stand-alone pipe supported on either side of the crossing by two new concrete abutments. The pipe was rigidly connected to the abutment with no flexible connection between the pipe and the bridge foundations (Tang and Cooper, 2009).

Fig. 5.4 groups the repair percentages for the four buried system components considered in this analysis (pipe, service laterals (IDU system), valves along pipes and valves along IDU systems). The data are relative to Zone 1, which corresponds to the area of the city of L'Aquila.
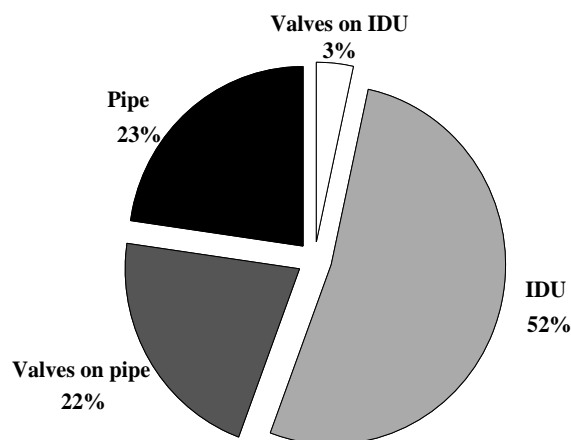
Fig. 5.4 Repair operations addressing damage to buried components for the portion of the gas network located in Zone 1: number of repairs for different buried components (Esposito et al., 2013)

The damage data were then analysed in terms of Repair Rate, RR, which is the ratio between the number of repairs and the length of the affected pipeline. Table 5.1 shows the repair rate for pipes, valves, and the two combined. Discontinuities and appurtenances accounted for a significant proportion in RR for the pipe, almost for all Peak Ground Velocity (PGV) values. The presence of in-line valves, service connections, and appurtenances has been identified as one of the factors increasing earthquake-induced damage to buried pipes. Repair rates were then evaluated with

respect to a pipe's geometrical and construction features. Material, joint type and pipe diameter are found to be the most important factors that influence the seismic vulnerability of buried pipelines (Tromans, 2004).

Table 5.1 Repair rate data evaluated for buried components with or without discontinuities

PGV (cm/s)	Length (km)	RR Pipes	RR in-line valves	RR pipes and valves
22	6.42	0.16	0.00	0.16
24	8.97	0.00	0.11	0.11
26	17.04	0.12	0.23	0.35
28	41.09	0.12	0.15	0.27
30	51.18	0.14	0.06	0.20
32	21.71	0.05	0.18	0.23
34	13.17	0.08	0.08	0.15

Socio-economic impact

Recovery of the gas network delivery started a few days after the earthquake. Data concerning the reinstatement of the gas service are shown in Fig. 5.5. The red line in the figure shows the percentage of customers that could have potentially been reconnected to the network, following the repair activities as a function of time. However, only a minor fraction of customers was reconnected quickly as the structural integrity of their housing needed to be ascertained first. From the figure it emerges that data on potential reconnection and end-user activation was available only from 6 May 2009 on. Consequently, from 6 April to 6 May 2009 a hypothetical trend for the network performance was assumed (dashed lines in the figure) considering that the entire network was shut off immediately after the earthquake.

Cascading effects

No cascading effects were reported.

5.1.1.4 Contributing factors

No contributing factors are known. The system was fully controlled during the emergency.
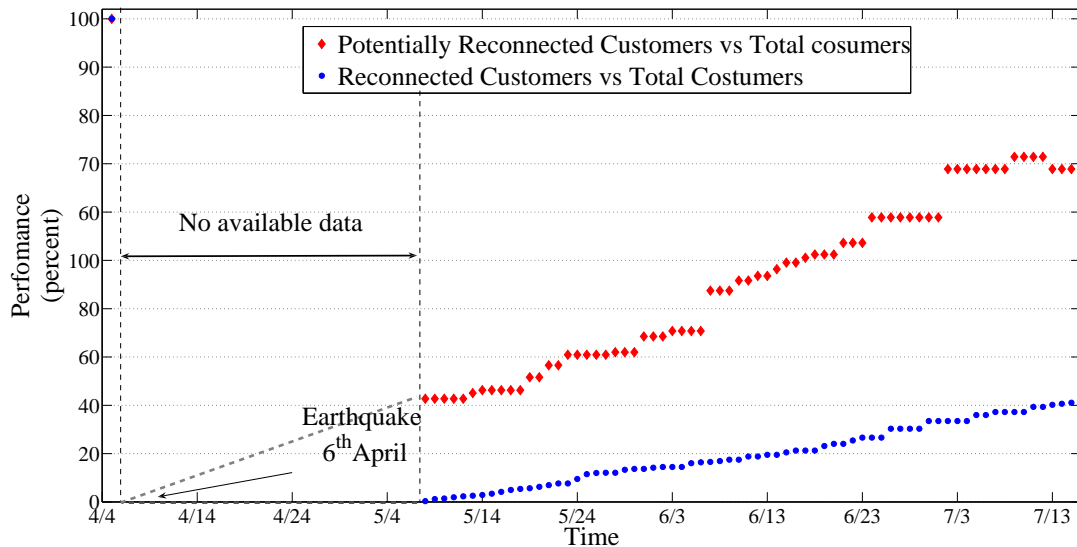
Fig. 5.5 Observed resilience-related curve for the L'Aquila gas network following the 2009 event (Esposito et al., 2013)

5.1.2   Northridge Earthquake, USA, 17 January 1994

The following sections present the impact of the earthquake on the gas network of the city of Northridge, which was subject to damage due to the lateral spread of liquefied soils. This case history is based on the work of Lau et al. (1995) and O'Rourke and Palmer (1996), and focuses on the performance of lifelines and/or pipelines.

5.1.2.1   Trigger characteristics

On 17 January 1994 an M_w = 6.7 earthquake occurred about 30 km northwest of Los Angeles, causing extensive damage to some major lifelines in the Los Angeles area. The maximum recorded PGA exceeded 1 g at several sites with the largest value of 1.8 g recorded at Tarzan, about 7 km south of the epicenter (USGS, 1994). In the vicinity of the Aliso Canyon gas storage field which is located approximately 13 km from the epicentre, PGA values of 0.62g were recorded. For additional general information on the Northridge earthquake the reader is referred to Section 2.1.3.

5.1.2.2   Impact dynamics

The service area of the Southern California Gas Company (SoCalGas) includes the epicentral area of the Northridge earthquake. The natural gas pipeline system of SoCalGas consists of 5,280 km of major transmission lines, 67,000 km of distribution mains, and over 67,200 km of service lines. The network serves a population of over 16 million in southern California. In the Los Angeles basin, which includes the earthquake epicentre, the local gas distribution system consists mostly of steel pipes of up to 40.6 cm (16 in) in diameter.

Damage and failure mechanisms

Detailed post-earthquake analyses showed that 242 metal gas pipe failures could be attributed directly to the earthquake. Of these, 35 occurred in major transmission lines consisting of metal pipes of up to 91.4 cm diameter that had been overstressed by more than 20% of the specified minimum yield stress of the pipe. The breaking of these gas transmission lines was caused by ground displacement resulting from the seismic waves travelling through the soil. Most transmission line failures were attributed to cracked welds (26 out of 35 cases). Other failure modes include leakage at the flange, buckling, and compression and tension failures, all associated with pre-1941 steel pipes of specified minimum yield strengths ranging from 170 to 240 MPa (O'Rourke and Palmer, 1996).

In the heavily impacted Los Angeles basin, the Northridge earthquake caused 123 distribution main and 84 surface line failures. Of the 123 distribution main failures, 62 cases were due to problems in the pipe welds while another 20 cases were caused by failures at flange and screwed connections. Only six cases were attributed to failure in the pipe body. The remaining cases were due to fracture failure of cast iron values or due to reasons not reported (Lau et al., 1995). Similar failure modes were identified with the service line system, although damage to the pipe body accounted for the majority of failures for this situation. In addition to the metal pipe failures, there were 33 incidents of plastic polyethylene gas line failures that were attributed to the earthquake.

Near the Aliso Canyon gas storage field a peak horizontal ground acceleration of 0.62g was recorded. But the actual PGA at the site could have been considerably higher because of amplification effects due to the higher elevation. Some pipes in the underground gas storage facility suffered damage due to the earthquake. The metal straps, anchor bolts, and concrete bases of many pipe supports for the aboveground gas transmission lines were damaged, as shown in Fig. 5.6 (Lau et al., 1995). The gas supply from Aliso Canyon was interrupted for five days (Schiff, 1997).

Gas lines above ground suffered significant permanent displacement after the earthquake because of landslides. In addition, the potential for landslides and stability failures due to earthquakes in the surrounding area are major concerns with respect to the exposed sections of previously buried gas lines (Lau et al., 1995). Many of these exposed pipes had lost their support because of soil movement due to the earthquake, as shown in Fig. 5.7. Figure 5.8 shows the shear failure of a buried pipe resulting from the lateral movement of the ground.

Figure 5.9 shows the failure in compression of a 55.9 cm (22 in) gas transmission line and an adjacent 106 cm (42 in) water line. Using the observed damage information it was estimated that the maximum earthquake-triggered compressive deformation of the surface soil at this location was about 30 cm (Lau et al., 1995).
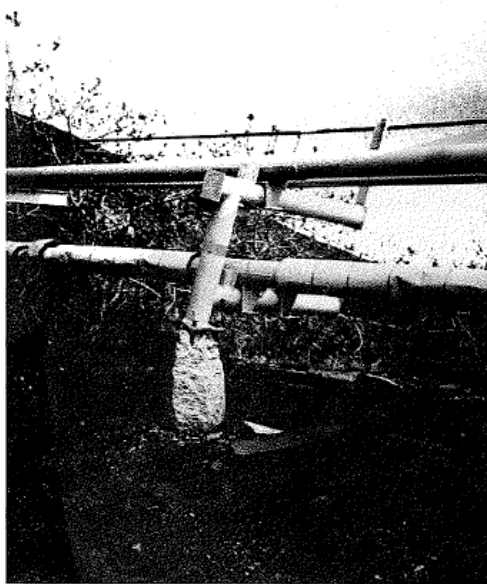
Fig. 5.6 Damaged support of aboveground gas line (Lau et al., 1995)

Fig. 5.7 Buried gas line exposed by landslide (Lau et al., 1995)

Fig. 5.8 Shear failure of gas line (Lau et al., 1995)

Fig. 5.9 Compression failure of gas and water lines (Lau et al., 1995)

Lau et al. (1995) also report that the same gas line failed in tension at the weld at another nearby location (Fig. 5.10). The tensile deformation of the ground at this location was estimated to be of the same order of magnitude as the compressive deformation. A little farther south of the tension failure point, the gas line failed initially due to compression, followed by tensile fracture, and then finally by buckling (Fig. 5.11).

Fig. 5.10 Tension failure of gas line (Lau et al., 1995)

Fig. 5.11 Cycled compression, tension and buckling failure of gas line (Lau et al., 1995)

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Protection measures and systems

Experience with the performance of gas lines during the Northridge earthquake shows that gas lines designed according to the then current seismic design standards generally performed well. In fact, near a damaged gas trunk line, a newer steel pipe which satisfied the API 5L piping standard, survived the earthquake with no noticeable damage. The performance of gas lines crossing fault lines was poor, and additional measures including special design considerations had to be implemented.

Many seismic gas shutoff valves tripped at industrial facilities, public buildings, etc. This mitigated the risk of fires by interrupting the flow of flammable gases.

5.1.2.3 Consequences

Damage to the critical infrastructure and its components

The earthquake caused damage and multiple ruptures in gas transmission and distribution lines, as well as in equipment at gas storage facilities. Releases of natural gas from broken pipes ignited in some cases, aggravating the consequences and causing concern due to the potential for spreading of the fire.

Socio-economic impact

The Northridge earthquake caused a total of 150,800 cases of customer outage, which represents 3% of the total meters in the SoCalGas system. Of these outages, 81% were initiated by the customers or emergency personnel, some apparently for precautionary reasons. For most of the cases (87.8%), service was restored later after inspection following the earthquake indicated no leakage or other damage problems.

Cascading effects

As a result of the fires caused by the accidental ignition of gas from ruptured gas lines many single-storey wood frame houses were destroyed. Fig 5.12 shows burning of the gas leaked from a ruptured gas line at a site near the Balboa Boulevard and Rinaldi Street intersection in Granada Hills where a 55.9 cm (22 in) diameter pre-World War II (1930) steel pipe was heavily damaged.

This site was located approximately 10 km north-east of the epicentre. A peak horizontal ground acceleration of 0.62g was recorded at a nearby location. The damage was evidently caused by the large ground displacement which occurred during the intense ground shaking of the earthquake. The damage covered several city blocks.

Fig. 5.12 Burning of leaked gas from ruptured gas line

5.1.2.4 Contributing factors

The age and corrosion level of some old pipelines contributed significantly to pipe damage and failures.

The earthquake not only impacted the natural gas network but also affected water mains and storage tanks, as well as fire stations. The fire departments stayed operational but had to resort to alternative water sources for fire fighting when the water supply failed. These sources would likely not have sufficed had conflagrations developed (Scawthorn et al., 1998).

5.2 LESSONS LEARNED FROM EARTHQUAKES

5.2.1 Systems weaknesses and critical components

Pipelines appear to be the weakest link in the natural gas storage and distribution network in case of strong ground motion and ground failure. Steel pipes generally show the highest vulnerability, constituting 72% of pipelines damaged during the L'Aquila earthquake. Old steel pipelines with oxy-acetylene weld joints are particularly susceptible to earthquake damage compared to electric arc welds, chemical welds and mechanical joints (O'Rourke and Palmer, 1996). Furthermore, small diameter (low pressure) pipes are more vulnerable to seismic loading than medium diameter (medium pressure) ones.

Even if natural gas distribution pipelines stay intact, gas meters can be damaged or destroyed due to debris impact from building collapse. During the Kocaeli earthquake, 860 gas meters suffered damage from collapsing buildings (Durukal and Erdik, 2008).

Natural-gas storage facilities are also subject to earthquake damage, in particular where tanks are supported by R/C columns, are not anchored or have hard pipe

connections. These connections are most susceptible to damage due to movement and rotation of the tank with respect to the supporting saddle.

5.2.2   Potential for propagation

In case of natural-gas releases, the ignition system of e.g. a truck is sufficient to ignite the gas. The risk of explosions or spreading of fires to surrounding structures, in particular in urban areas, can be significant. In situations like these it is of paramount importance that the gas network be immediately shut down to avoid any escalation of the event.

5.2.3   Consequence severity and extent

Consequences can range from damage to distribution and storage equipment resulting in service outage to fires following gas leaks. Reconnection to the gas network can be slow as repairs or replacements will likely be required at multiple locations, and the post-earthquake structural integrity of buildings needs to be ascertained before commercial activities or households can be reconnected. If the gas ignites, the severity of the accident depends on where the leak occurred. In an urban environment, consequences can be more severe and extended.

Contemporary earthquake damage to fire-fighting structures or the breaking of water mains, can constitute a problem in case of fires in the natural-gas network. The vulnerability of emergency resources to the earthquake needs to be assessed in earthquake-prone areas.

5.2.4   Protection measures and systems

Past earthquakes have shown that pipe materials and joint detailing critically influence the resistance of the natural-gas system to earthquake loading (Lanzano et al., 2013). Newer pipelines made of polyethylene are more resistant to earthquake impact compared to steel pipes. During the Northridge earthquake there were only 27 repairs in polyethylene distribution pipes, with over 24,000 km of polyethylene piping in service at the time of the earthquake (Schiff, 1997). Construction of pipes according to the latest design standards is the most effective protection measure.

In case of gas leaks due to earthquakes, seismic gas shutoff valves will interrupt the flow of fuel that otherwise would continue unabated, supplying fuel to support explosions or major fires. Automatic gas shutoff systems are usually used as a protection measure in gas storage-distribution plants.

Above-ground natural-gas storage in tanks is subject to the same potential problems during earthquake loading as tanks found in other industrial activities. Seismic base isolation can be an effective method for the protection of liquid storage tanks against earthquake loading depending on the characteristics of the ground motion and slenderness of the tank (Abali and Uckan, 2010). Other protection measures are discussed in in Section 2.2.

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5.3 INDUCED SEISMICITY

Induced seismicity refers to minor earthquakes that are triggered by human activity, such as fossil-fuel or groundwater extraction, mining, construction of reservoirs etc. The impact of induced seismicity on structures in general and critical infrastructures in particular is not routinely assessed and little information on the risks exists.

Seismicity induced by hydrocarbon exploration is observed mainly in the northern part of The Netherlands, where we find besides several smaller gas reservoirs also the Groningen reservoir, one of the world's largest gas reservoirs containing a reserve lasting at least several decades. The induced events are characterized by shallow, small events occurring in or in the close vicinity of exploration fields. Since 1986 the Dutch seismological monitoring network of the KNMI (Royal Dutch Meteorological Institute) has been observing induced events. Occasionally, earthquakes of up to M_L = 3.5 have caused minor damage (such as cracks in buildings). More often the felt events are of general annoyance to the local population. Since 1 January 2003, the new Dutch mining legislation requires for each concession a hazard analysis and monitoring plan. Within the context of this new mining law site-specific engineering hazard parameters are being estimated, i.e. ground motions that can be associated to specific risks.

With very little first-hand experience available on the hazards associated with induced seismicity, further research is needed to get a better grasp of the potential risks involved with respect to critical infrastructure damage.

6 Ports

6.1 EARTHQUAKES

6.1.1 Hyogo-Ken Nanbu (Kobe) earthquake, Japan, 17 January 1995

6.1.1.1 Trigger characteristics

The Hyogo-Ken Nanbu (Kobe) earthquake (M_w=6.9, M_{JMA}=7.2 Richter) occurred at 5:46 a.m. (local time) on 17 January 1995. The rupture was initiated at a depth of approximately 10 km at a distance of roughly 20 km south-west of downtown Kobe. Kobe is located in Osaka bay which is the second greatest urban and industrial area in Japan, after the Tokyo area. The earthquake lasted approximately 20 seconds, enough to cause many injuries, deaths and damage to buildings and infrastructure. Overall, the earthquake resulted in more than 6,000 fatalities and over 30,000 injuries. Fires following the earthquake incinerated the equivalent of 70 US city blocks. They together destroyed over 150,000 buildings and left about 300,000 people homeless. The economic loss was estimated at about 200 billion US$ (NIST, 1996).

The Kobe earthquake had a probability ranging from 0.4 to 8% over 30 years when the earthquake occurred in an area of a complex faulting system in 1995 (Shimazaki 2001). The strike-slip rupture propagated bi-laterally from the hypocentre, with the rupture to the east extending directly beneath Kobe City. The length of the seismic fault is estimated to be 40 to 60 km, 9 km of which were activated and caused the earthquake. The maximum horizontal surface displacements reached 1.7 m, while the corresponding vertical displacements were 1.3 m (Somerville, 1995).

The combination of directivity effects and the close proximity of Kobe to the fault rupture caused extremely strong shaking. The strong motion in the Port of Kobe reached peak ground acceleration levels of 0.54g and 0.45g in the horizontal and vertical directions, respectively. The duration of the strong motion segment was about 10s. Liquefaction and lateral spreading were also observed (Toki, 1995).

6.1.1.2 Impact dynamics

Ports contain a wide variety of facilities for passenger operations and transport and generally consist of waterfront structures, cargo handling and storage components, and various infrastructures (buildings, utilities, transportation infrastructures etc.). From an engineering point of view, ports are soil-structure systems that consist of various combinations of structural and foundation types. Typical types of seismic damage to port structures are (PIANC, 2001):

- Damage to gravity quay walls,

- Damage to sheet pile quay walls,

- Damage to pile supported wharves,

- Damage to cranes,

- Damage to breakwaters,

- Damage to buildings,

- Damage to all kinds of lifeline systems (utility systems and transportation infrastructures).

These damage types can be caused by ground shaking or liquefaction phenomena. However, the liquefaction of loose, saturated, sandy soils that often prevail at coastal areas (especially reclaimed land an uncompacted fills) is the most widespread source of seismic damage to port infrastructures.

Damage and failure mechanisms

The Kobe earthquake caused major damage to the infrastructures the port of Kobe. Most of the quay walls are of rigid block type made of concrete caissons. The caissons had been placed on top of gravel fill consisting of decomposed granite which had completely replaced the soft clay layer beneath the caisson for improving the bearing capacity and reducing settlements. The most severe damage occurred in those caisson walls that (a) were nearly parallel to the coastline (and thus parallel to the causative fault), and thereby experienced the stronger fault-normal accelerations (Somerville, 1998); and/or (b) had been designed with a small seismic coefficient of 0.10 to 0.15. In contrast, the caisson wall of the main wharf at Maya Futo, designed conservatively with a large seismic coefficient of 0.25 and running almost perpendicular to the fault (and thereby having been subjected to some less severe accelerations parallel to the fault), did not experience any visible damage or substantial deformation, remaining operational after the earthquake.

Liquefaction settlements at both Kobe Port and Rokko Island ranged from 30 cm to 50 cm throughout the harbour. Quay walls moved up to 5 m toward the sea. The walls also settled about 1 to 2 m and tilted about 4 degrees toward the sea. The damaged was caused mainly by deformation in the loosely deposited foundation soil beneath the caisson wall (Khodabakhshi and Baziar, 2010). Residual horizontal displacements (RHD) of the caissons based on a survey are shown in Fig. 6.1 (Inatomi et al., 1997). The underground utilities of the pile supported buildings and transportation corridors were subjected to damage.

During the earthquake, significant tilting or rotation of walls in addition to horizontal deformations were observed, reflecting cyclic bearing capacity failures of wall foundations during earthquake loading (Figs. 6.2 and 6.3). Concrete caisson quay walls suffered substantial outward displacement and rotation (Fig. 6.4). The caissons did not overturn (Hamada and Wakamatsu, 1996; Inagaki et al., 1996; Kamon et al., 1996) which renders their overall performance better than that of anchored sheet-pile walls which in earlier earthquakes, that were much less devastating than the Kobe

earthquake, frequently experienced collapse (e.g. Kitajima and Uwabe, 1978; Gazetas et al., 1990).

Figure 6.5 shows a schematic of the various damage modes of a typical caisson-type quay wall during the earthquake (Na et al. 2008).
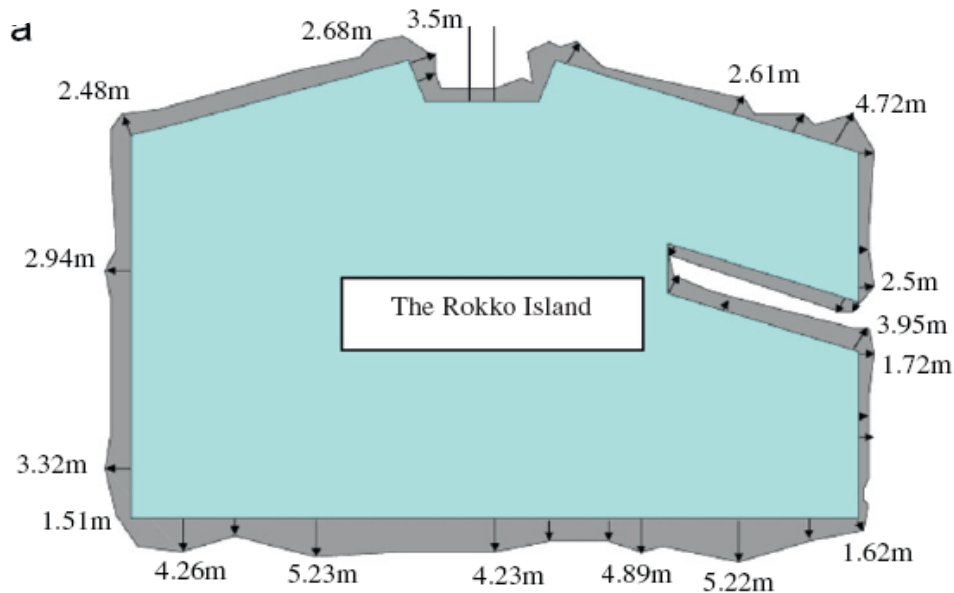
Fig. 6.1 Field observation of RHD for damaged quay walls at the Rokko Island in Kobe port (Inatomi et al., 1997)

Fig. 6.2 Extremely extensive damage to apron pavements
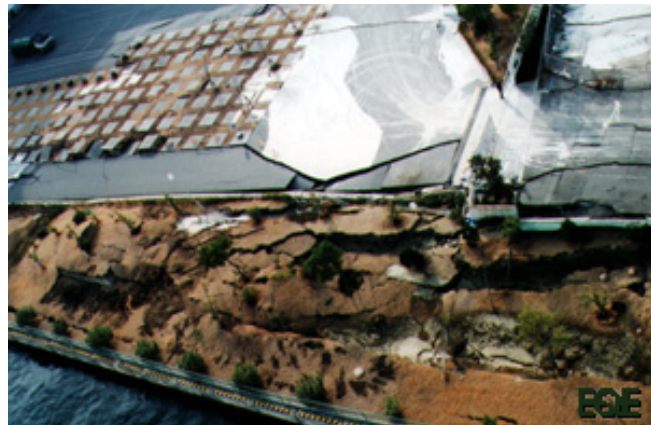
Fig. 6.3 Lateral spreading, liquefaction and settlement along the shore of the Port of Kobe

Fig. 6.4 Seaward displacement and tilting of quay-walls
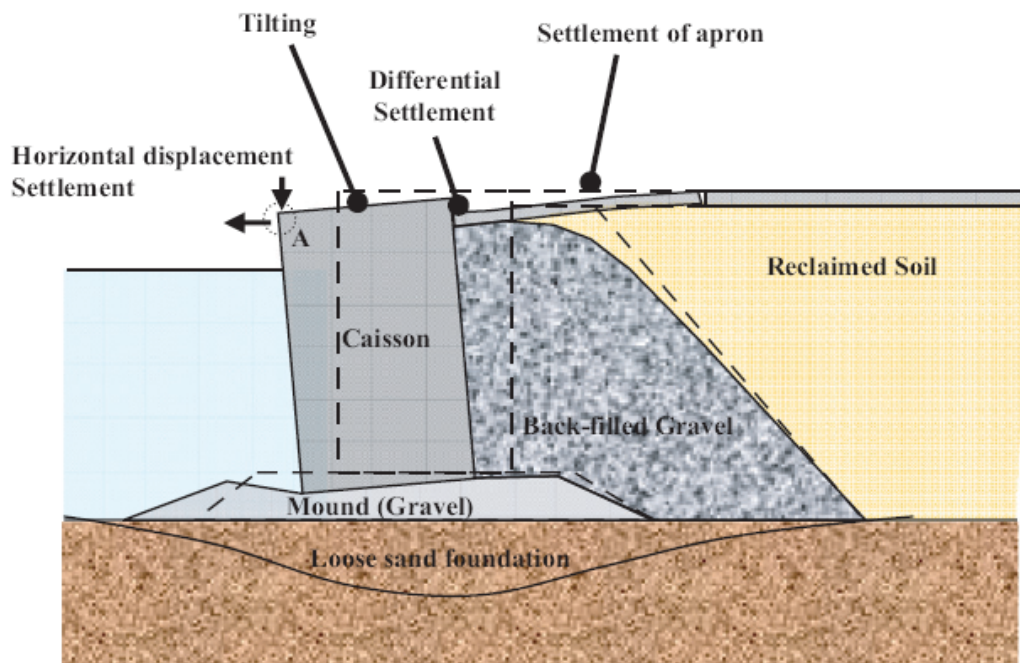
Fig. 6.5 Typical damage modes for gravity type quay walls and parameters for damage criteria (Na et al., 2008)

The large diameter pile supported Takahama wharf in the port of Kobe (Fig. 6.6) suffered horizontal displacements between 1.3 and 1.7 m. The deck of the wharf was made of reinforced concrete slabs and beams supported by steel pipe piles with a diameter of 700 mm. These steel pipes buckled at the pile heads except for the piles with short out of soil height, located more towards the land. Cracks were found at the pile cap to concrete beam connection located most landward. In certain piles buckling also occurred below the mud-line. The backfill behind the retaining structure settled about 1 m. Buckling of piles was observed close to the boundary between soil foundation layers of different stiffness. The level at which the thickness of the piles were reduced because of factory weld joints, happened to be close to the boundary

between these materials. Such a phenomenon was not observed in longer piles located most seaward.

Fig. 6.6 Damage to a quay-wall at the port of Kobe (Nozu et al., 2004)

The cellular pile quay-wall of the Maya wharf No.1 at the port of Kobe suffered warping due to non-uniform displacements. The net horizontal displacement varied between 1.3m and 2.9m. The vertical displacement varied between 0.6m and 1.3m. Consequently, the cellular quay wall tilted a maximum angle of 11 degrees (Borg, 2007).

With respect to cargo handling equipment out of a total of 55 cranes, 22 were slightly damaged (Level II damage) by the Kobe earthquake and 30 cranes were seriously damaged (Level III damage). Characteristic failure modes of cranes during the earthquake are shown in Figs. 6.7 – 6.10. A classification of the damage is shown in Table 6.1 (Tanaka and Inatomi, 1996).

The causes of crane damage in Kobe Port are attributed to the lateral deformation of the caissons that carried the seafront rails of the cranes and the rocking vibration of the cranes, which resulted in excessive section forces on the legs and local buckling (Tanaka and Inatomi, 1996). During the earthquake, container crane supports failed, also resulting in significant damage. In some cases the crane rails spread until plastic hinging developed in the portal frame (Soderberg et al., 2009). One crane collapsed due to excessive spreading.

Fig. 6.7 Seaward lateral movement of rail foundation and differential settlement of rails (Rokko Island)

Fig. 6.8 Crane total collapse. Various damages to other cranes like plastic hinges and bending of their members (Rokko Island)

Fig. 6.9 Crane damage due to deformation of foundation soil

Fig. 6.10 Sand emersion near container cranes (Port Island)

Table 6.1 Classification of damages to cranes (Tanaka and Inatomi, 1996)

Leg Span	30m		16-20m			Total	Damage Level
Location	RI	PI	RI	PI	M		
Level I	0	0	0	0	0	0	Only derailment
Level II	4	0	1	11	6	22	Slightly damaged legs
Level III	14	7	4	4	1	30	Seriously damaged legs
Level IV	0	0	0	0	1	1	Damaged girder
Level V	1	0	0	0	0	1	Completely destroyed
Obscurity	-	-	1	-	-	1	
Total	19	7	6	15	8	55	
RI: Rokko Island, PI: Port Island, M: Maya Wharf							

An overview of the measured ground motion and the regions of highest damage at Kobe Port is provided in Fig. 6.11.
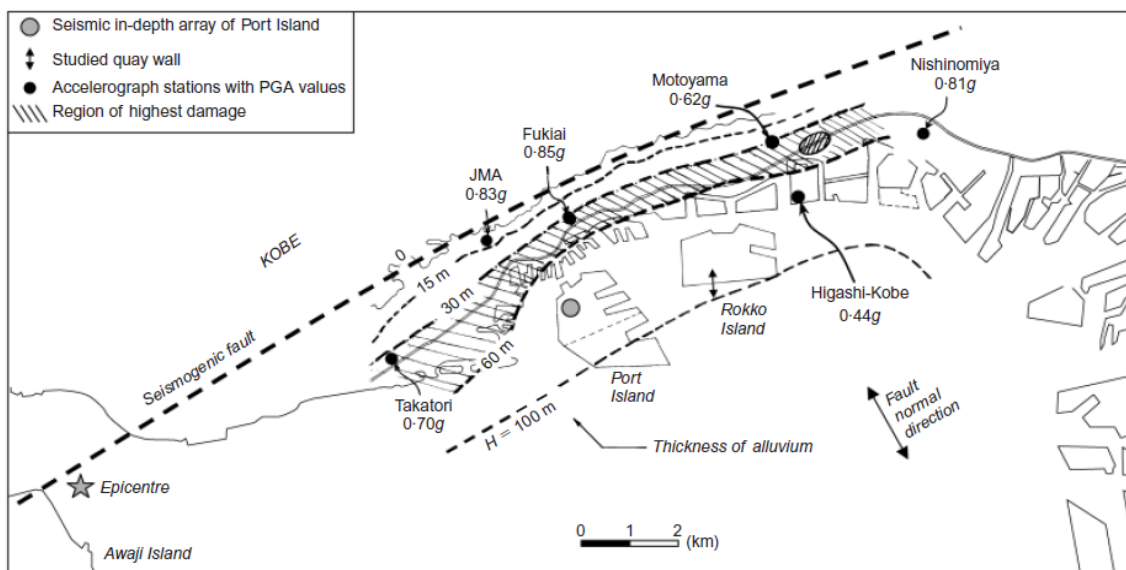
Fig. 6.11 Map of Kobe showing the seismologically inferred earthquake fault and acceleration stations showing the recorded PGA. The shaded region shows the area of highest structural damage (Dakoulas and Gazetas, 2008)

Protection measures and systems

The port of Kobe was built almost exclusively on reclaimed land. Sandy soil (often hydraulic fill) has been placed over the soft clay deposits resulting in significant settlement due to the consolidation of clays. The loose nature of the fill resulted in an extremely high susceptibility to liquefaction. Soil improvement techniques, such as preloading to minimize differential settlements under structures, sand drains, sand compaction piles and 'composite' piles were implemented in the interior areas of Port Island. Very little soil improvement was performed at the shipping berths and wharves along the periphery of the islands (Werner and Dickenson, 1996).

Liquefaction was one of the main reasons for the extended damage at Kobe port during the earthquake, along with the facilities' design lacking to consider realistic seismic effects on inertia and ground loads. These minimum measures and other means of passive protection of port facilities against a strong earthquake were not implemented.

6.1.1.3 Consequences

Damage to the critical infrastructure and its components

The main components of the port that were affected by the earthquake were gantry cranes, quay walls and the wharves of the port area. Most of the quay walls at Kobe Port which are made of concrete caissons suffered severe damage due to liquefaction effects. Damage to cranes could be attributed mainly to movement of the

rail foundations caused by ground failure and resulting in bending of their members. As a consequence, loading and unloading of cargo from ships was essentially brought to a standstill by the damage to the dockside cranes associated with the horizontal displacements of the rail systems tat was running on the quay walls.

Socio-economic impact

Transport infrastructures, like ports, are critical because they involve large investments, a long process of rebuilding, and possibly a lower priority in rebuilding than other vital systems (OECD, 2003). Furthermore, after a catastrophic event, such as a strong earthquake, port facilities can contribute significantly to recovery activities during the crisis period, particularly in cases of excessive damage on other means of terrestrial transportation (Na et al., 2009). Studies on the impact of the 1995 Kobe earthquake (Chang, 2000a) on maritime activity indicate that such delays can entail a persistent loss of activity in areas where competition originating outside the disaster area is strong. Therefore local and regional economic losses resulting from such events can be deep and persistent.

The Kobe earthquake destroyed 90% of the 187 berth-port (the fourth largest container port in the world) and the economic and social costs were staggering. The earthquake resulted in extended closure of the port of Kobe, while direct physical losses (cost of repairs/restoration) amounted to US$ 5.5 billion (in 1995 monetary value). However, the overall economic losses exceeded US$ 6 billion during the first nine months after the earthquake (Werner et al., 1997; Werner, 1998).

The damage at the port of Kobe was so extensive that even 2 years after the event the rehabilitation work had not been completed. Fig. 6.12 shows the recovery in the port (restoration expressed in terms of the facilities that are in operation as a percentage of total port facilities before the earthquake, or port traffic as a percentage of the corresponding traffic on the month of the earthquake) (Chang, 2000a). In 1999, the Hyogo Prefectural Government reported that the Port of Kobe had only recovered 80.4% of its monthly amount of exports and imports as compared to before the earthquake. This permanent loss of business occurred even though the port had recovered 75% of its cargo-handling capacity one year after the earthquake.

The long-term indirect losses due to closure of the port and the diversion of the incoming traffic, although difficult to quantify, are even higher. According to Chang (2000b), the ship owners were forced to invest in other ports during Kobe Port's recovery, and the majority did not return. Fig. 6.13 shows the movement of containers in selected ports of the Asian continent during 1992-1997 (Chang, 2000b). The sharp drop in freight traffic for the port of Kobe is obvious, during the year of the event (1995) and for the following 2 years. Even more than ten years after the earthquake, the total number of containers in the port of Kobe is still lower than those in 1994 and the port of Kobe fell from the 6th position of major freight traffic ports in the world prior to the earthquake to the 38th position in 2006 (AAPA, 2009). This demonstrates that the port's operations from a business competition

point of view were not sustainable under an earthquake intensity such as the one of the Kobe earthquake.
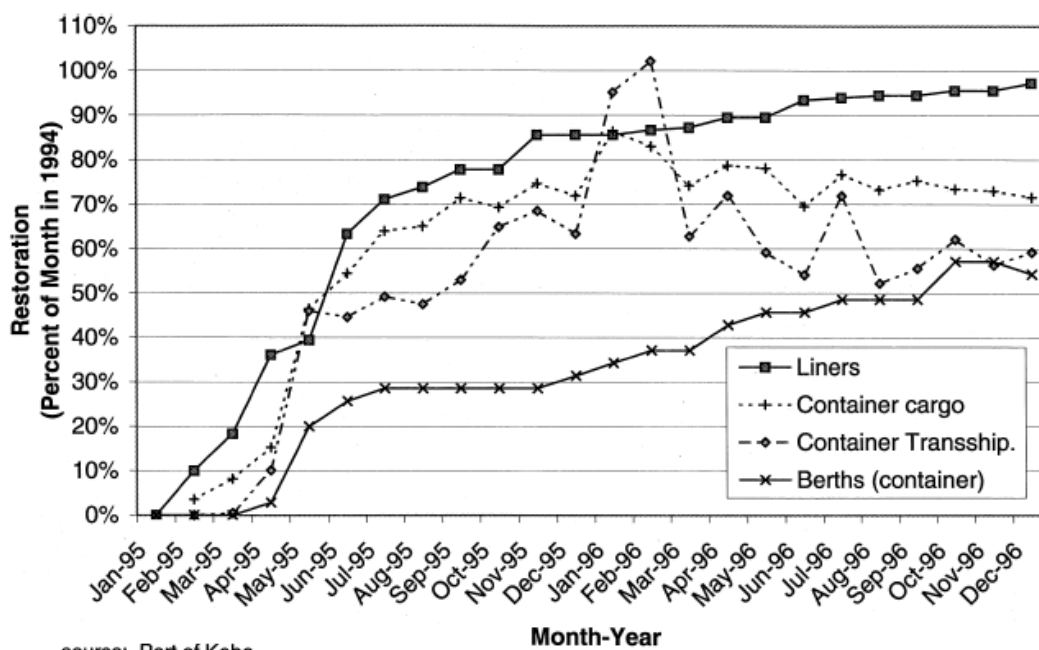
source: Port of Kobe

Fig. 6.12 Restoration of the port of Kobe after the Hyogo-Ken Nanbu 1995 earthquake (Chang, 2000a)
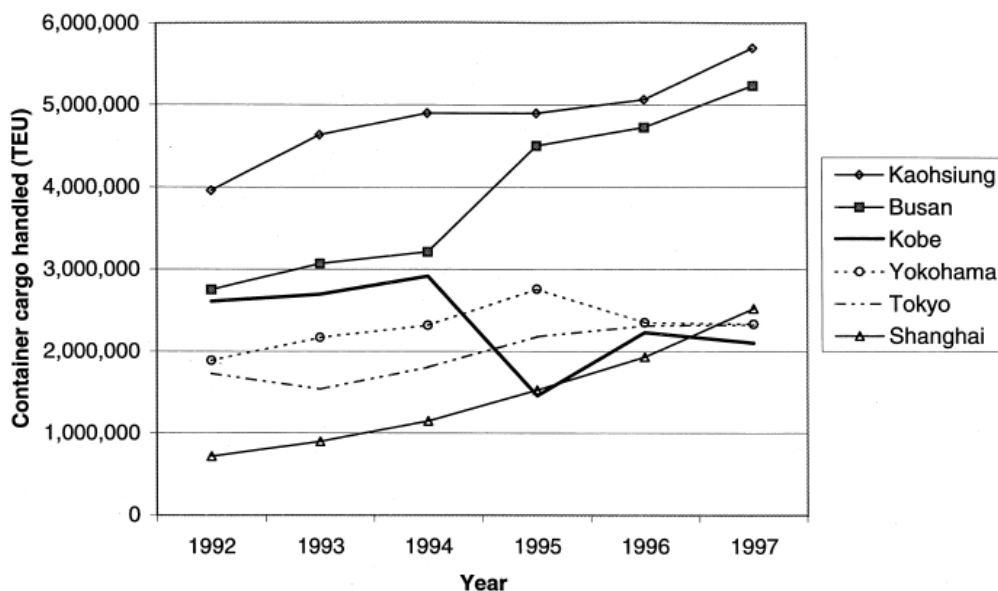
Fig. 6.13 Ship cargo traffic in Asia during the years 1992-1997 (Chang, 2000b)

Based on the experience with the Kobe earthquake, insurance companies consider that a large earthquake in Greater Tokyo would entail damage of between US$ 1,000 billion and US$ 3,000 billion, equivalent to 25%-75% of Japan's GDP (OECD, 2003).

Cascading effects

Extensive cascading effects and system interactions occurred in the Kobe earthquake. A characteristic example is the case of a 25 km quay wall which sustained damage involving outward displacement of about 1-3 m (sometimes even up to 5 m) as a result of liquefaction having taken place in the soil behind or possibly underneath the wall. This outward displacement successively propagated rearwards, bringing about varying amount of damage to storage tanks and industrial facilities located there (Sivathasa et al., 2000). The observed lateral displacements were generally large near the water front and decreased with distance inland.

The earthquake impact at Kobe Port caused damage to electric power, gas, water-works, telecommunication and transport networks, as well as to administration infrastructure (Tsuruta et al., 2008).

6.1.1.4 Contributing factors

The Kobe Earthquake reemphasized that moderate to large earthquakes directly beneath (or close to) densely urbanized areas can cause catastrophic loss of life and property as well as serious damage and economic loss to port facilities. Important factors contributing to these losses were the proximity to the earthquake crustal-rupture zone, amplification effects of soft-soil deposits and liquefaction susceptibilities of reclaimed land and soft soil deposits.

Furthermore, preservation of the accessibility of port facilities with transportation systems is of utmost importance in case of a crisis. The observed damage in transportation systems, mainly highways and bridges, caused serious problems for accessing the port of Kobe after the 1995 earthquake (Werner, 1998).

6.1.2 Great East Japan (Tohoku) earthquake, Japan, 11 March 2011

6.1.2.1 Trigger characteristics

The M_w 9 Tohoku earthquake is the largest earthquake ever recorded in Japan, causing death and destruction over a wide swatch of land. PGA values were high in the north part of Sendai City where soft surface layers deposit on the hard rock (Motosaka, 2012).

The port of Sendai is located approximately 64 km from the rupture zone of the Tohoku earthquake. The shaking intensity MMI in the region was VIII (USGS, 2011). Instrumentation located several kilometres away from the port indicated very strong ground shaking with PGA levels ranging from 0.64g at the Sendai Station to over 2.0g at the Shiogama Station. The >2.0g PGA measured at Shiogama was among the highest values recorded for the Tohoku earthquake main shock (Percher, 2014).

General details on the Great East Japan Earthquake characteristics are provided in Section 2.1.1.

6.1.2.2 Impact dynamics

Damage and failure mechanisms

The main facilities of the port of Sendai are the Takamatsu Wharf, the Takasago Wharf, the Raijin Wharf and the JX Nippon Oil Refinery, as shown in Fig. 6.14. The Sendai Port wharf characteristics are given in Table 6.2 (Percher, 2014) and individual damage mechanisms are discussed in the following.
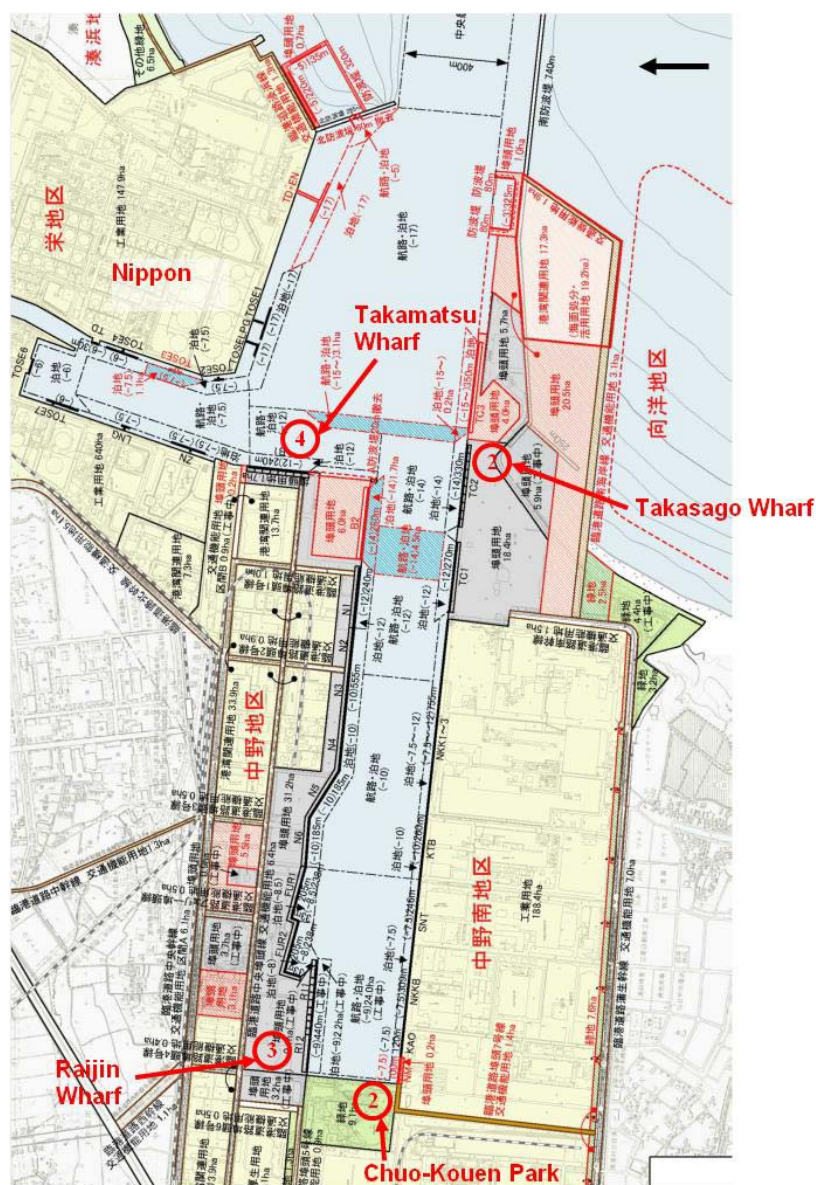
Fig. 6.14 Sendai Port (Percher, 2014)

Takasago wharf

Compared to the rest of the facilities in the port of Sendai, the eastern port of the Takasago wharf performed poorly with localized failure of the wharf structures and large ground deformations. Generally, the structural damage was concentrated on

the reclaimed land where liquefaction was prevalent. Very little damage was observed in portions of the facility developed by excavation and little reclamation with import, hydraulically-placed fill. Liquefaction occurred in the loose fills during the March 11 main shock, and was repeated in subsequent aftershocks, as evidenced by reports of sand boils in the container yards during a major aftershock on 7 April 2011 (Percher, 2014).

Table 6.2 Sendai Port wharf characteristics (Miyagi Prefecture, 2011)

Wharf	Berth Number	Water Depth (m)	Length (m)	Maximum Vessel DWT (m ton)	Year Built	Construction Type
Takasago	1	-12.0	270	30,000	1995	Sheet Pipe Piles
	2	-14.0	330	50,000	2001	Sheet Pipe Piles
Raijin	1	-7.5*	130	5,000	1977	Sheet Pipe Piles
	2	-7.5*	130	5,000	1979	Sheet Pipe Piles
	3	-7.5*	130	5,000	1979	Sheet Pipe Piles
Takamatsu	1	-12.0	240	30,000	1973	Bulkhead

*Note: Water depth was increased to 9.0 m in year 2010

The performance of the waterfront facilities, including bulkheads, cranes, and crane rail and supporting beams, was found to be directly related to the ground deformations throughout the facility. In the western section of the terminal it appeared that damage due to liquefaction and ground deformation was minimal, based on only minimal pavement damage and absent tsunami scour. The structural configuration and stiffness of the anchored bulkhead, combined with the well-drained backfill of cobbles and boulders contributed to the very good seismic performance (Percher, 2014).

Surface evidence for ground deformation increased substantially across the eastern portion of the wharf, which had been developed with hydraulically placed fill. The pattern of pavement cracks and structural displacements suggests that the eastward trending lateral spreading extended back into the terminal roughly 200 m from the "free-face" at the boat harbour, as shown in Fig. 6.15. In this zone it appears that the lowest resistance to ground movement was toward the boat harbour where the closure bulkhead wall along the east side of the wharf rotated and translated outward, as shown in Figs. 6.16 to 6.18. This rotation resulted in a significant ground crack about 15 meters inboard of the closure wall (Percher, 2014).

Approximately 1.5 m of vertical settlement was observed. While the majority of the structural damage at this location was likely due to inertial loading during strong shaking and kinematic loading associated with lateral spreading, the subsequent tsunami appears to have scoured significant amounts of soil from underneath the

end of the crane rail girders exposing the supporting steel pipe piles, tie rods, and the back face of the bulkhead (Percher, 2014).

Fig. 6.15 Lateral spreading in the backlands along the original shoreline (Percher, 2014)

Fig. 6.16 Rotated wharf closure bulkhead at the east end of the Takasago wharf (Percher, 2014)

Fig. 6.17 Significant vertical settlement near the east end the Takasago wharf (Percher, 2014)
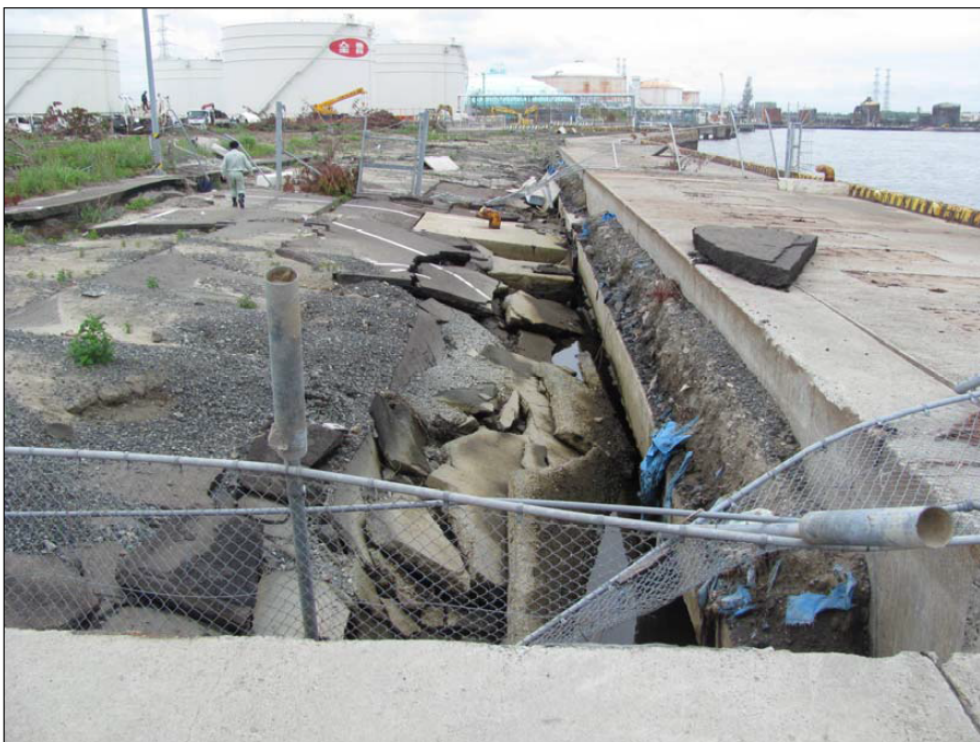
Fig. 6.18 Ground failure behind the quay wall at Sendai port (TCLEE, 2012)

The Takasago wharf is located on the beach ridge. Liquefaction was observed only near the sheet pile quay wall of the No. 2 pier between two piers, probably because

the background was constructed by filling with crushed rock and split stone, whereas the No. 1 pier was constructed by concreting the natural ground. Differential settlement between the crane rail, supported by steel piles, and the ground surface was about 10 cm in the No. 1 pier, as shown in Fig. 6.19, whereas it reached 88 cm in the No. 2 pier (Yamaguchi et al., 2012; Percher, 2014). Settlement of the ground, movement of the quay wall toward the sea, and opening of cracks in the backyard of the quay wall were observed.

Raijin Wharf

Raijin Wharf, located at the north-western end of the port, is a loading dock used to service factories in the vicinity. The wharf extends approximately 390 m toward an adjacent wharf to the east, Nakano ferry terminal, with an average water depth of 9 meters. The original wharf structure is of sheet pipe pile construction with steel strands connected to deadman anchors with batter piles in the backland. The central 220 m of the structure was retrofitted in 2005 with additional sets of batter piles and steel ties to increase the dredged depth while improving the seismic performance. After the earthquake and tsunami, the wharf structure appeared undamaged. Although it is unclear what the performance would have been without the new tie rods, the retrofit worked well (Percher, 2014).

From a geotechnical standpoint, the good performance of this facility may stem from the apparent absence of liquefaction in this area. It appears that this portion of the port is within an area excavated by dredging and significant fill is not expected to exist. As a result, the subsurface conditions are more likely to consist of dense native soils, which are less susceptible to liquefaction than typical waterfront fills (Percher, 2014).

Fig. 6.19 Differential vertical movements of the crane rail girders and settlement of fill surface (Percher, 2014)

<u>Takamatsu Wharf</u>

The Takamatsu Wharf, located at the north-eastern end of the port and across from the JX Nippon Oil refinery, is a tied back bulkhead structure extending approximately 250 m with an average water depth of 12 m. This sheet pipe pile construction was completed in 1982. It appears to be constructed in an area developed by dredging into native ground. This is supported by the general lack of evidence of liquefaction in the area and the location of the original shoreline well to the east. The loose backfill soils, where existed, within the anchorage zone between the bulkhead and the deadmen have been improved using chemical grout. The wharf face generally remained straight, except for the south corner, which was not improved by chemical grouting and where bulkhead movement was observed (Percher, 2014).

<u>Performance of cranes</u>

The cranes of Sendai Port performed well in general. The minor permanent deformations of the Takasago wharf resulted in differential horizontal movements between the construction joints of the crane rails, in response to the lateral spreading thereby, as shown in Fig. 6.20 (Percher, 2014).

The cranes without base isolation survived the earthquake and tsunami with no apparent structural damage. The base isolated cranes, using either springs and vertical dampers or rubber isolators and shear pins, also survived the earthquake and tsunami. There is also evidence that the isolation system that was designed for 0.2g lateral forces worked well during the earthquake. No derailment occurred. The cranes did not experience any crane-vessel interaction as no vessel was berthed at the terminal at the time of the earthquake. Numerous scratches and damaged non-structural elements on the structures were visible and indicated possible tsunami debris impact (Percher, 2014). The spreader of one of the cranes seems to have fallen on the crane during the earthquake or the tsunami as shown in Fig. 6.21 (TCLEE, 2012).

The base isolated crane rail structure moved during the earthquake (Fig. 6.22). There is evidence that part of the rail did not move in phase during the earthquake. If a crane had been in place at the time of the earthquake it would have been subjected to lateral displacement in its base. In the future design of a crane, the potentially differential movement of the crane rails should be considered (Percher, 2014).

Fig. 6.20 Landside crane rails at the eastern side of Takasago Wharf (Percher, 2014)
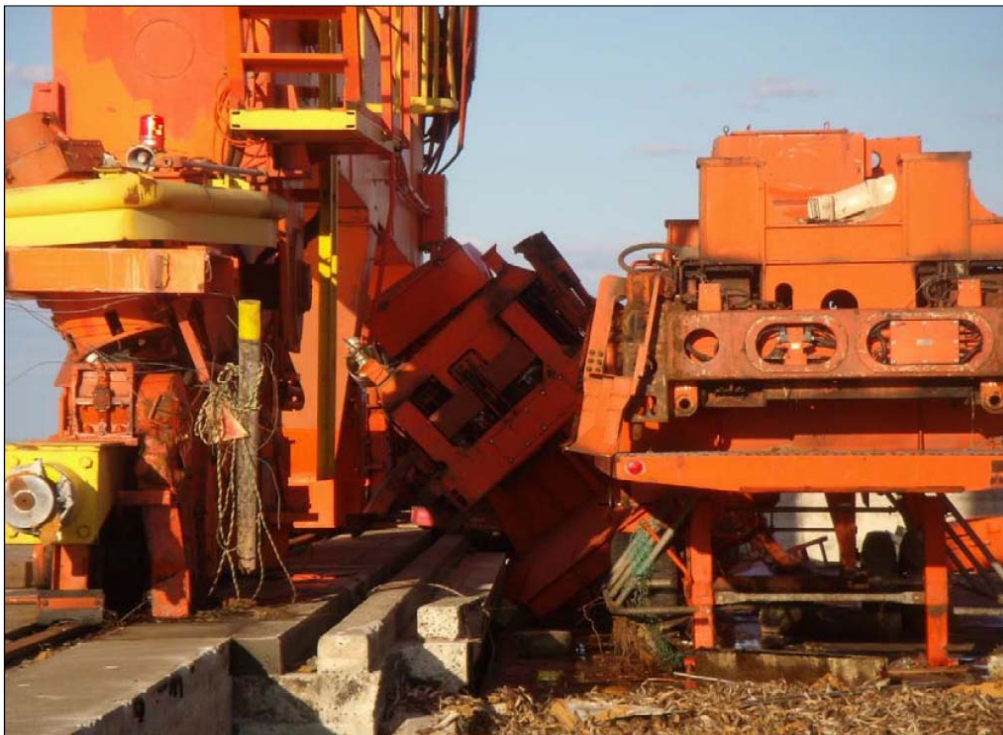
Fig. 6.21 Crane damage at Sendai Port (TCLEE, 2012)

Fig. 6.22 Movement of waterside base isolated crane rails (Percher, 2014)

Protection measures and systems

An earthquake event of the intensity of the 2011 Tohoku earthquake was not considered in the design of structures. Fig. 6.23 shows pseudo velocity response spectra (damping 5%) in the NS and EW directions at several buildings near Sendai Station compared to the design spectra. The figure shows that the design period content is exceeded significantly at several stations (Motosaka, 2012).
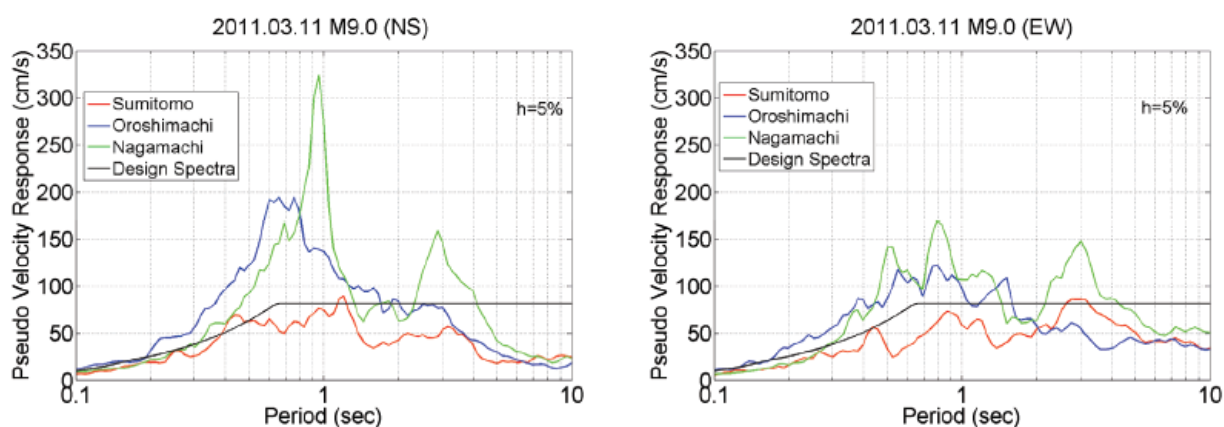
Fig. 6.23 Comparison of velocity response spectra due to different soil conditions (Motosaka, 2012)

An earthquake early warning system is available since 2007. While this system allows individuals to protect themselves in various environments such as houses, offices and factories, the warning lead time is not sufficient to mitigate impacts on industrial activities and equipment.

6.1.2.3 Consequences

Damage to the critical infrastructure and its components

Damage to the port infrastructures due to ground shaking was limited. Most damage was caused by liquefaction effects that led to the localised failure of wharf structures. Damage to bulkheads, cranes, crane rails and supporting beams was observed. Wharfs that had been hardened to earthquake impact, e.g. by adding tie rods or with chemical grouting, performed significantly better. While some damage to cranes was observed, in general they performed well.

Socio-economic impact

The port of Sendai is one of the largest port facilities in North-East Japan, and one of 23 Japanese ports that have been classified as especially important ports by the Japanese Ministry of Land, Infrastructure, Transportation, and Tourism (Percher, 2014). These ports are considered to have significant impact on the regional and national economy. Sendai Port is a multi-purpose port facility comprising a container terminal, commercial loading/unloading wharves, and a ferry terminal. The construction of the port dates back to 1967, and the port has grown tremendously in the last 40 years. Major port construction occurred between 1969 and 1979, resulting in the 2.5 km long channel the port is centred on today. The container terminal opened in 1990 and has seen steady growth in the number of handled containers each year. In recent years, the terminal served roughly 185,000 TEU (twenty-foot equivalent unit) annually (Percher, 2014).

Although part of the port facilities was damaged during the earthquake, (mainly pier and bulkhead movement), the Takamatsu Wharf remained intact and played an important role in servicing emergency response vessels in the Sendai area (Percher, 2014).

Cascading effects

No cascading effects were reported due to the earthquake. At the JX Sendai oil refinery located at Sendai Port, minor hydrocarbon releases were triggered by the earthquake but did not constitute a major risk (Krausmann and Cruz, 2013). However, the tsunami that hit the port after the earthquake triggered considerable damage. The impact of the tsunami on Sendai Port is presented in Section 6.3.1. The tsunami damage to JX Sendai oil refinery is discussed in detail in Section 2.3.1.

6.1.2.4 Contributing factors

An earthquake of this magnitude was not expected and hence its intensity not considered in the design of structures. An updated seismic hazard assessment was prepared following the Tohoku earthquake (Fujiwara and Morikawa, 2012).

The main factor that greatly affected the performance of the port facilities after the 2011 Tohoku earthquake was the tsunami that hit the seafront structures and caused damage to port structures, facilities and general operations. Damage due to ground shaking was rather limited.

6.1.3 Kocaeli earthquake, Turkey, 17 August 1999

6.1.3.1 Trigger characteristics

The reader is referred to Section 2.1.2 for general information on the Kocaeli earthquake characteristics. This section deals with the observed damage at port facilities and the consequences of the Kocaeli earthquake on marine structures, ports and jetties.

6.1.3.2 Impact dynamics

Most of the port facilities damaged by the Kocaeli earthquake are within 10 km of the fault rupture and lie in intensity zone IX. Marine structures in Izmit Bay were affected severely by the Kocaeli earthquake (Fig. 6.24). The structures on the southern shores of Izmit Bay are mainly on alluvial and marine deposits and those along the northern shores of the bay are on relatively stiff soil (Durukal and Erdik, 2008).

Damage and failure mechanisms

The majority of the ports and jetties privately operated by industrial facilities in Izmit Bay sustained damage. This included the failure of piers, mechanical equipment and piping, and the collapse of cranes (Durukal and Erdik, 2008). The earthquake showed that port and harbour facilities are particularly susceptible to submarine landslides or ground settlement due to the liquefaction that can occur during earthquakes. Some of the port structures suffered pounding damage during the earthquake.

Haydarpasa Port in Istanbul, located about 60 km away from the closest fault break, suffered minor damage to quay walls. The quay walls of Tuzla Port, located about 25 km northwest of the closest fault break, moved about 40 cm horizontally and the backfill settled about 10 cm. Ground failure was observed near the jetty entrance of the port facility of the Petkim petrochemical plant. This port was not operational afterwards. Many of the battered piles beneath the jetty were badly damaged. Some of the pipelines along the pier fell off their supports and were damaged. Ground cracking and deformations were observed along the shoreline near the pier. The failure of the jetty and the elevated pipe way of the Tupras Refinery prevented the

loading and unloading of all fuel-oil products at the refinery. The jetty was composed of a reinforced concrete deck that was supported on steel piles in-filled with concrete. The middle half of this pier sagged due to damaged piles. Ground deformations and cracking along the shoreline were observed near the pier. A substantial number of jetties at industrial facilities were also damaged including Petrol Ofisi, Shell Oil, Trans Turk, Seka Paper Mill, Public Marina at Izmit, Fursan, and UM Shipyard.
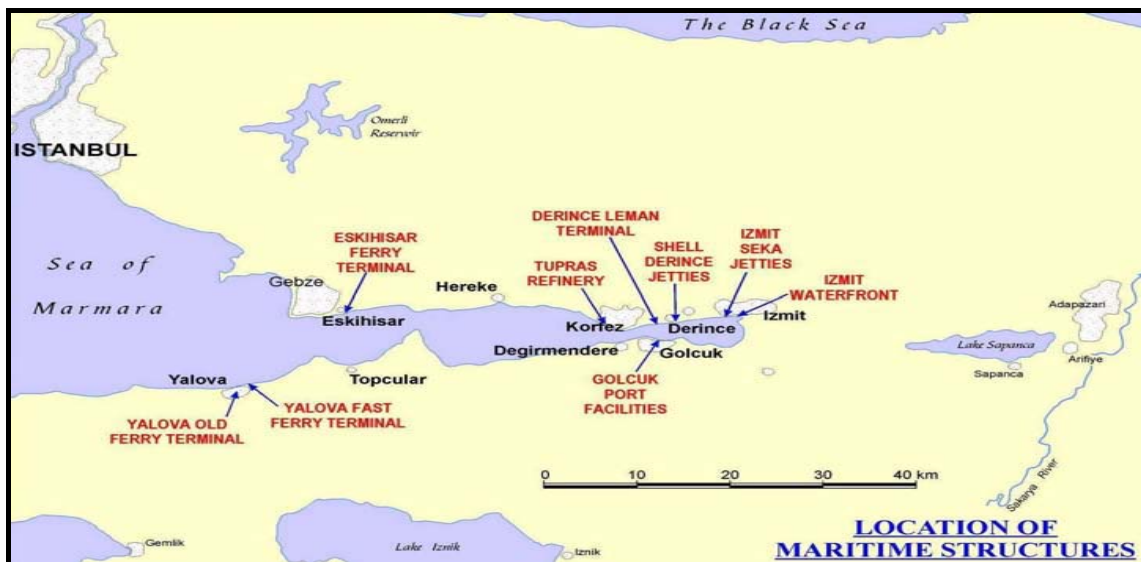
Fig. 6.24 Location of port structures around Izmit Bay

Derince and Gölcük ports also suffered heavy damage to docks, cranes and warehouses, including cracks and severe subsidence (Fig. 6.25 and 6.26). Extensive damage was observed at fault crossings, e.g. at the navy base. It included failure of steel piers and piping systems and the collapse of cranes. In Derince, a general cargo and grain port handling about 2 million tons of cargo annually suffered heavy damage to docks, cranes and warehouses, including cracks and severe subsidence. The concrete caisson type bulkhead, with a length of about 1.5 km, shifted away from the wharf up to 0.7 m horizontally and 1 m vertically, due to liquefaction-induced deformations, settlements and lateral spreading. Two of the three rail mounted main portal cranes (RTGs) were non-functional and some old steel warehouses were damaged.

The jetty of the (un)loading facility at PETKIM petrochemical plant was damaged due to movement of the wharf (Suzuki, 2002). A new wharf constructed on piles did not suffer any problems. A displaced steel piped jetty in Izmit and a damaged jetty at Gölcük Naval Base are shown in Fig. 6.27. Although some failures could be attributed to age and poor construction, the failure in Gölcük Naval Base showed that even a modern structure with excellent construction can be vulnerable if the design is inappropriate. Another problem was due to differential settlement and different stiffness of the quay structures. It caused damage due to pounding between the structures.
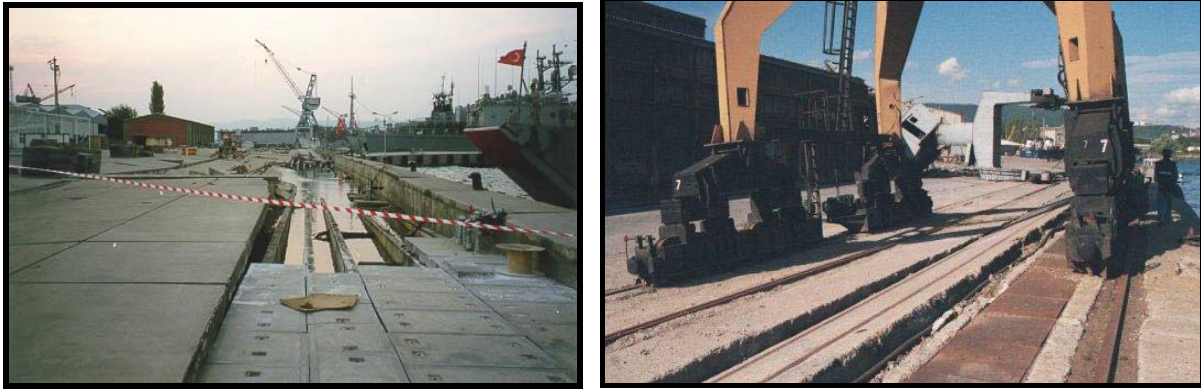
Fig. 6.25 Collapse of ground slabs due to settlement at the Navy port in Gölcük (left) and damage at Derince port (right)

Fig. 6.26 Damage at the navy port in Gölcük (left) and failed column at SEKA port (right)

Fig. 6.27 A displaced steel piped jetty in Izmit (left) and a damaged jetty at Gölcük Naval Base (Bilham et al., 2003)

Protection measures and systems

No special protection measures or systems seemed to have been implemented.

6.1.3.3 Consequences

Damage to the critical infrastructure and its components

Ports suffered extensive damage to nearly all facilities. This included damage to quay walls, docks, warehouses, piles beneath jetties and pipelines. Cranes were also affected.

Socio-economic impact

The earthquake affected a wide, heavily-industrialised area. The impact on commercial activities was therefore significant. The total estimated loss for port facilities in the region was estimated to be in the order of US$ 200 million (EERI, 1999). In addition to damage to structures, business operations were disrupted as (un)loading operations of goods were not available due to damage.

Cascading effects

No cascading events were observed for port structures.

6.1.3.4 Contributing factors

No contributing factors are known.

6.2 LESSONS LEARNED FROM EARTHQUAKES

6.2.1 Systems weaknesses and critical components

Strong earthquakes can affect all port facilities, ranging from quay walls, docks, warehouses and cranes to piles beneath jetties, pipelines and industrial manufacturing and storage activities. Evidence from past earthquakes suggests that most damage to port structures is associated with significant deformation of a soft or liquefiable soil deposit. Furthermore, most damage occurs due to excessive deformations and not catastrophic collapses. Past experience has demonstrated that even moderate levels of earthquake intensity can cause liquefaction (PIANC, 2001).

Damage can be severe because in many cases at the time of construction of major port facilities, neither all the important material seismic response behaviours, such as soil liquefaction, nor a realistic approach of the seismic loading in terms of imposed acceleration and ground inertia forces on structures might have been considered in the analysis and design. Even if design regulations changed to account for these phenomena, a more consistent soil-structure interaction mechanism in some cases may be needed (Tanaka and Inatomi, 1996).

It is interesting to note that during the Kobe earthquake caisson-type quay walls with identical configurations, similar soil conditions and located at the same site with a similar seismic intensity, experienced different degrees of damage. It was

acknowledged that inherent variations of soil properties and construction processes exist, requiring an improved quality control (Na et al., 2008).

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 foundations caused by ground failure, resulting in bending of their members (PIANC, 2011). Cranes that are restrained or anchored to foundation rails are subjected to inertia forces, like any other structure with rigid foundation connections, rendering them vulnerable to failure due to bending and ground shaking. 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 can induce damage to adjacent structures and other facilities. In addition, the performance of cranes can be affected due to settlement and/or the horizontal movement of foundation rails due to liquefaction of subjacent soil layers. Rail de-alignment can result in damage to wheels (ATC-25, 1991). Today's larger jumbo cranes are more susceptible to earthquake damage than early container cranes that are lighter and hence would be subjected to lower seismic forces in the crane structures. 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.

Administrative buildings located at the port are subject to shaking damage, as well impacts caused by loss of bearing or lateral movement of foundation soils.

6.2.2   Potential for propagation

In the Kobe earthquake traffic congestion caused by paralysis of highways, etc. and breakage of lifelines caused by bridge collapses seriously hampered the emergency functions of hospitals, administrative services and also port functionality. This shows the need for a well prepared recovery strategy that takes into account the damage propagation among interdependent infrastructures in a massive urban disaster (Tsuruta et al., 2008).

Analysing interdependencies between infrastructures during earthquakes, Tsuruta et al. (2008) highlight that ports can either be affected by the disruption of other infrastructures or their closure can impact other infrastructures. Interruption of the electric power and gas supply, waterworks, telecommunication and transport networks will have adverse effects on port operations. Conversely, the closure of a port due earthquake damage can affect gas delivery and administration infrastructures.

In case of earthquake-triggered hazardous-materials releases at industry sited at the port, fires and explosions can occur that could propagate to adjacent facilities if adequate separation distances are not observed.

6.2.3   Consequence severity and extent

Past earthquakes highlight the seismic risk to port facilities. Extensive damage due to ground motion, or soil liquefaction and lateral spreading can lead to prolonged closure times with potentially serious impacts on the regional economy when customers start diverting their business to alternative ports. This can have repercussions to long after when the port operations have been completely restored. In addition, the degree to which a key infrastructure can function after a catastrophe determines the rate at which materials and men can be moved to the affected region. This in turn affects the post-event productivity of the region (Murlidharan and Shah, 2003).

With respect to the resilience of lifelines based on lessons learned from the Kobe earthquake, Casari and Wilkie (2004) indicated that the timing of lifeline repairs has a considerable impact on social welfare and that decentralised decisions by lifelines firms are not socially optimal. However, economic incentives can change the behaviour of lifelines firms, or could be used to set different priorities and speed up the reconstruction of lifelines. It was also suggested that at no extra cost the government could introduce a payment scheme that induces firms to incorporate into their decisions the full social costs and benefits of their repairs following a natural disaster.

6.2.4   Protection measures and systems

An effective solution to protect port facilities from earthquake impact is to harden their critical elements. The seismic design of the structures and infrastructure components of a port should consider, as far as possible, a realistic evaluation of the seismic behaviour of the materials in situ and the imminent seismic loads as well as the interaction between all the elements of the facilities/infrastructures (PIANC, 2001). This requires, of course, a sound assessment of the seismic hazard.

After the Kobe earthquake it was recognized that, although the pseudo-static analysis for caisson quay walls is effective for the serviceability-level design ground motion, it is not applicable for the Level II design ground motion, which is a safety-level design ground motion, because of its higher intensity. Thus a need was recognized for a more sophisticated analysis in which seismic performance of a quay wall beyond the limit of force balance can be assessed. For this reason, the effective-stress analysis for quay walls was incorporated in an updated version of the technical standard for port and harbour facilities in Japan (Ministry of Transport, 1999). This new standard also includes pushover analysis for pile-supported wharves.

Where liquefaction can be an issue, implementing appropriate remediation measures against the phenomenon can effectively increase the seismic performance of port structures. At Sendai's Port's Raijin wharf, the original sheet pipe pile wharf structure was retrofitted in 2005 by adding batter piles and tie rods which improved the wharf's

seismic performance. At Takamatsu wharf the loose backfill soils were improved using chemical grouting with good results during the Tohoku earthquake.

Strategic decisions can be made to design important port operations to resist higher earthquake loading than others. When reconstructing the harbour facilities of Kobe Port it was decided to design liner berths at selected locations for higher seismic loads (Kameda, 2000).

After the Kobe earthquake in 1995, Japan invested US$ 1 billion in research and development of an earthquake early warning system. The country's meteorological agency implemented the system in December 2007. The system automatically issues a warning if a severity threshold is exceeded which will help save lives. Given the very short warning lead times, its applicability for shutting down port operations or industrial activities is, however, limited.

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6.3 TSUNAMIS

6.3.1 Great East Japan earthquake tsunami, Japan, 11 March 2011

6.3.1.1 Trigger characteristics

The tsunami triggered by the Great East Japan earthquake affected a wide swath of land and caused a large number of fatalities, injuries, and destruction of infrastructures. At Sendai Port, the inundation height varied between 4 and 7 m maximum, as shown in Fig. 6.28 (Percher, 2014).
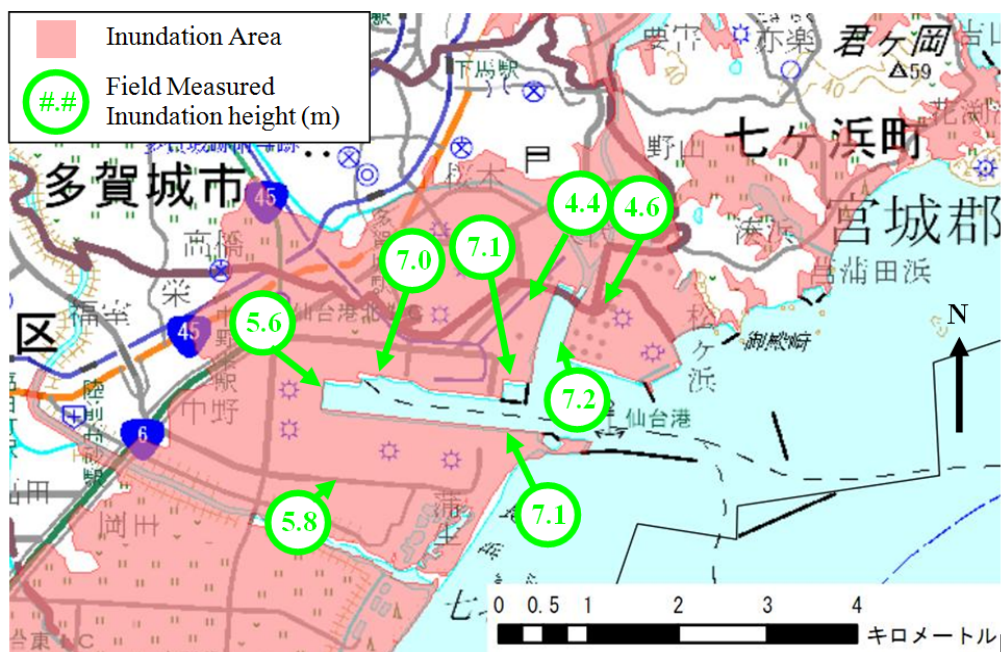
Fig. 6.28 Inundation heights in Sendai Port (Percher, 2014)

6.3.1.2 Impact dynamics

The tsunami height is a function of the shore profile and geomorphology. Since ports have relatively large water depths the tsunami does not break. In extreme cases it can overtop quay walls and seawalls and flood the area with a rapid current. In addition to damage or destruction of coastal defences, and transport networks, tsunamis of more than 10 m height can impact ports through (Takahashi et al., 2011):

- Drifting and collision of ships,

- Destruction and inundation of port facilities including industry,

- Drifting and collision of timbers and containers,

- Debris deposit in ports,

- Scouring and deposit in ports.

The main forces acting on coastal structures by tsunamis are 1) pressure and uplifting, 2) drag forces and 3) scouring. Scouring is the most typical failure mode for coastal structures by tsunami.

Damage and failure mechanisms

During the Great East Japan earthquake and tsunami, port structures were subjected to the external forces of the seismic motion and the subsequent tsunami. Whether the cause of the damage was one or the other, or a combination of causes, is not clearly known at present. Generally speaking, breakwaters (including tsunami barriers) were mainly damaged by the tsunami, and quay walls were mainly damaged by the seismic motion and liquefaction. Also, the effect of the tsunami on the ports in the northern Tohoku Region was large, and the effect of the earthquake force was large south of Ishinomaki and Sendai Ports (Kazama and Noda, 2012).

In the Port of Sendai the amount of damage compared to other facilities was not as severe (TCLEE, 2012). In Chuo-Kouen Park at Sendai Port, according to information provided by an eyewitness, the retaining wall failed during the earthquake shaking (PARI, 2011). Significant additional damage likely occurred due to the tsunami hydrodynamic force, which was concentrated by the long narrow path along the shipping basin and abruptly focused on the failure area. Satellite imagery of the failure area indicates that the wall debris was washed over a large area within the basin (Percher, 2014), which is an effect of the wave drawback force that was concentrated in that area.

During the tsunami, many ships broke their moorings and collided with port facilities and one was washed on a wharf at Sendai Port (Fig. 6.29). According to the Maritime News Paper, a total of 6 vessels of 20 to 200 thousand tons were stranded or caused oil spills within ports.

The severe damage to warehouses and factories in industrial areas at ports industry areas caused secondary impacts to industry. Container terminals suffered from inundation and more than 4,000 containers at Sendai Port floated from their foundations and were scattered (Tomita and Yoem, 2012). Of these, 1,000 went into the sea (Fig. 6.30). These containers added to the tsunami debris.

Many breakwaters and seawalls were also damaged by the tsunami. A total of 14 major ports were affected (PIANC, 2001).

Protection measures and systems

Japanese ports have an earthquake/tsunami protocol in place to minimise port damage due to errant vessels. In many cases, this protocol is to stop operations, evacuate non-essential personnel, and evacuate vessels where possible. Even with these preparations, it is clear that many vessels were caught off guard and could not escape the tsunami (Percher, 2014). Overall, six large vessels of 20,000 to 200,000 dead weight tons (DWT), 31 passenger ships, and an estimated 17,000 small boats were stranded, damaged, or caused oil spills (Takahashi et al., 2011).

Fig. 6.29 Container ship swept ashore by tsunami at Sendai Port and damage on crane (TCLEE, 2012)
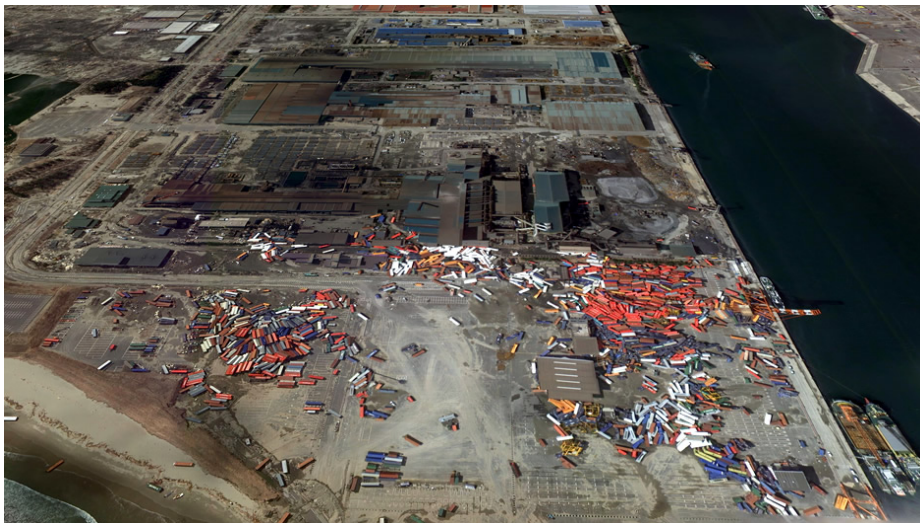
Fig. 6.30 Scattered containers Sendai Port after the tsunami (Tomita and Yoem, 2012)

Tsunami early warning systems were in place but updates on the severity of the event were not timely enough for areas close to the earthquake epicentre. Also, offshore and onshore tsunami barriers existed due to the proneness of the Tohoku region to tsunamis (Mori and Takahashi, 2012). However, many barriers were overtopped or did not withstand the onslaught of the tsunami waters. The design of breakwaters and seawalls should assume overtopping and resilience (Koshimura, 2012).

Tsunami hazard maps are useful tools for understanding risks to an area and hazard maps were available for the Sendai area. However, more often than not hazards are

underestimated, and Fig. 6.31 shows a comparison of inundation areas according to the hazard map and actual affected areas.
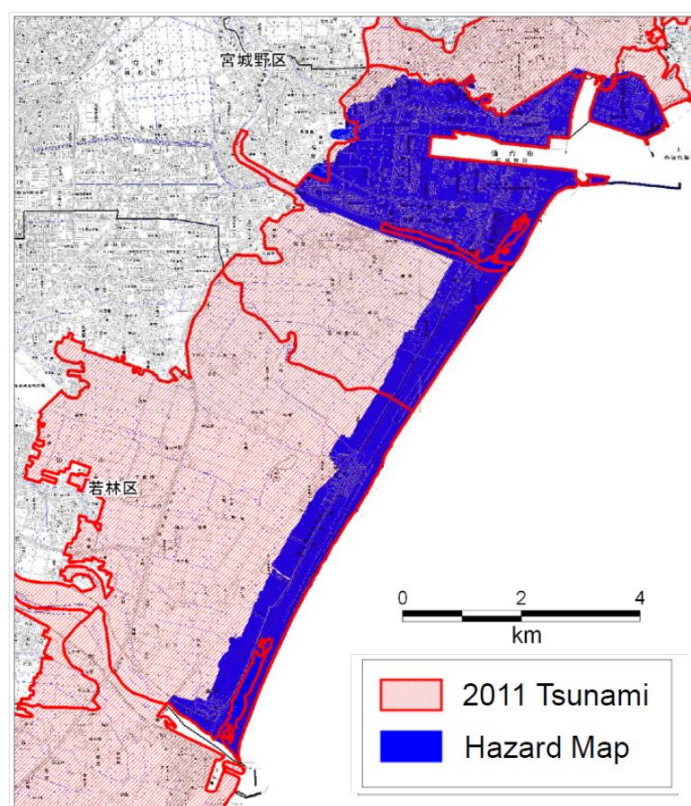
Fig. 6.31 Sendai tsunami inundation hazard map (Koshimura, 2012)

6.3.1.3 Consequences

Damage to the critical infrastructure and its components

The tsunami affected all port infrastructures and caused structural and non-structural damage. Port facilities and components affected were seawalls and breakwaters, quay walls, ships, (un)loading terminals, warehouses including stock, and industry located in the port. In addition to the sheer impact of the tsunami on structures, damage due to collision with the debris-laden waters was also observed. The debris that was swept along with the tsunami also caused blocking of the port's access roads.

Socio-economic impact

Many ports were seriously damaged by the tsunami but reopened to limited ship traffic by 29 March 2011 (Imamura and Anawat, 20112). Also Sendai Port suffered extensive damage and was temporarily closed. It reopened on 16 April 2011 when the commodity distribution system was completed (Kyodo News, 2011).

The tsunami caused the death of ten personnel at the port. Due to the proximity to the earthquake's epicentre, evacuation times were limited. Also, no evacuation order was given by the port authority immediately after the earthquake.

Economic damage due to tsunami impact on transportation infrastructures (including ports) are quantified as 27 billion US$ (EERI, 2011).

Cascading effects

The tsunami triggered hydrocarbon releases at the JX Nippon Sendai oil refinery which ignited, causing a major fire and endangering other facilities at the port, as well as emergency response resources (Krausmann and Cruz, 2013). The impact of the tsunami on the Sendai oil refinery is discussed in detail in Section 2.3.1.

Following the earthquake and tsunami, Sendai Port's Takamatsu Wharf was available to service emergency response vessels in the Sendai area. However, the fire at the adjacent Sendai oil refinery made it difficult for vessels with emergency supplies to enter the Sendai port facility. Two days after the extinction of fire on 15 March, "Kaisho Maru", the first emergency response vessel in Sendai area, berthed at Takamatsu Wharf and unloaded much-needed supplies (Percher, 2014). Had it not been for the fire in the Oil Refinery, the vessel would have arrived two days earlier in the area. Had the retrofit of the wharf not been successful, this facility would not have been available to transfer these critical supplies.

6.3.1.4 Contributing factors

Two coastline types exist along the coast of Tōhoku, the rias in the north and the plains in the south of Sendai. These different topographies have significant effects on an incoming tsunami. The shape of the river valleys in the rias coastline constraints the incoming waves and amplifies the tsunami's run-up height, but restricts the inland inundation of the waves. In contrast, the coastal plains offer lower resistance to the incoming waves which limit run-up height but allow further inland inundation of the tsunami than the rias (Chian et al. 2012).

In view of the frequent occurrence of tsunamis along the Tohoku region, coastal defences had been constructed along vulnerable coastlines in Taro, Kamaishi and Ofunato. However, the design height of coastal defences based upon historical earthquakes was considerably lower than the Tohoku earthquake tsunami in 2011 (Chian et al. 2012).

Studies of the early warning system response show that there was a delay in time to truly understand the scale of the event and the unprecedented size of the resulting tsunami. The initial estimates made 3 minutes after the event predicted wave heights of 1 m to 6 m. Updates increasing the predicted wave heights occurred at 28 minutes and 44 minutes; however, by this time the initial, and in some cases maximum, tsunami wave had already hit the worst devastated areas (Percher, 2014). It is unclear how beneficial the later prediction updates were as electrical power and cell

phone contact may have been lost at that time, and many people were already moving to higher ground (Percher, 2014).

In Sendai Port, approximately 1,000 personnel were on site at the terminal when the earthquake hit. Facility staff stated that while there was no loss of life from the shaking, the tsunami resulted in the death of ten personnel at the facility. Because of the proximity to the epicenter, the personnel had only limited time to evacuate. There were only approximately 45 minutes before the largest tsunami wave reached the coastline near Sendai. Additionally, there was a several minute delay between the earthquake and the tsunami warning issued by the Japan Meteorological Agency. While tsunami warning sirens sounded, there was no official evacuation procedure or direction given by the port authority, therefore evacuation decision making was done on an individual or team level basis. Most personnel evacuated to tall buildings in the vicinity and survived the tsunami; however, a small number of personnel made the decision to evacuate the area in vehicles and did not survive (Percher, 2014).

6.4 LESSONS LEARNED FROM TSUNAMIS

6.4.1 Systems weaknesses and critical components

Many tsunami protection measures were implemented in Japan but these measures were insufficient against the 2011 event. The Tohoku tsunami was unprecedented in inundation height and spatial extent and easily overtopped all defensive breakwaters and seawalls, barriers were severely damaged, some reinforced concrete buildings were totally destroyed, and inundation maps were severely underestimated in several areas (Mori and Takahashi, 2012; EERI 2011). It is not straightforward to separate the damage at ports caused by the Great East Japan earthquake and by the tsunami it triggered, but there are indications that breakwaters are very vulnerable to tsunami damage while quay walls are more susceptible to ground motion and liquefaction.

All port facilities are at risk from tsunami impact due to their heightened exposure compared to other infrastructures. Non-anchored equipment or pieces of cargo can be moved or carried away with the tsunami waters, becoming debris that can collide with other structures and cause significant damage. In fact, ships that have broken their moorings due to tsunami inundation and began drifting have been identified as a major source of damage risk in port areas. Drifting ships caused various problems during the Tohoku earthquake either because of damage to the ships themselves, collision with other structures, and because they can constitute and obstacle for restoration. Therefore, it is important for disaster prevention purposes to predict the drifting motion and route of a large ship driven by tsunami current (Suga et al. 2013).

6.4.2 Potential for propagation

Cascading effects from tsunami impact at port facilities would be mostly economic, caused by the disruption of the supply chain from damage and facility downtime. The

generation of disaster debris (e.g. from cargo containers or cars swept away by the tsunami waters) can exacerbate the damage due to collision with structures or equipment.

Many ports are home to manufacturing or storage activities involving hazardous materials. The risk of releases from these industries during a tsunami is high. Should flammable substances be accidentally released by tsunami impact, they can ignite and the fires can spread with the tsunami waters over wide areas, increasing the risk of accident propagation to other parts of the port. Lessons learned from tsunami impact at an oil refinery in a port area are discussed in Section 2.4.

6.4.3 Consequence severity and extent

Generally, the estimation of the global consequences and their extent is directly related to the intensity of the tsunami and the preparedness measures taken. Extreme events can produce major damage and losses even if countermeasures had already been implemented prior to tsunami impact. However, it is not clear, yet, to which extent existing mitigation measures helped reduce the damage. At present, there is no cost-benefit analysis of the level (and cost) of countermeasures against an extreme low probability event.

The availability of a tsunami early warning system should help avoid major human losses at ports, provided that evacuation times are sufficient to reach higher ground and timely evacuation orders are given by the port authorities. The main tsunami losses at ports would then be caused by possibly unavoidable damage to structural and non-structural components, and the associated downtime of port operations.

6.4.4 Protection measures and systems

While the most effective solution to avoiding tsunami impact on critical assets are sensible land-use planning decisions, high economic stakes are often associated with port operations in certain regions. Therefore, tsunami damage to port structures may be unavoidable and outside of the range of rational design criteria (Eskijian, 2008).

Although the Tohoku region was in principle well prepared for tsunamis in general, the Great East Japan earthquake tsunami highlighted that structural measures cannot completely prevent tsunami disasters. However, a study commissioned by The World Bank found that while many dikes and breakwaters were destroyed by the tsunami, they were nevertheless somewhat effective in mitigating damage (Ishiwatari and Sagara, 2012). The study also concludes that it is unrealistic to build protection structures large enough to safeguard people and assets from the largest conceivable events. Instead, the resilience of conventional structures should be enhanced to mitigate damage even when the design hazard level is exceeded. The possibility for structures to "fail gracefully" should also be explored as a mechanism to delay the tsunami impact and dissipate some of its energy. Ishiwatari and Sagara (2012) argue

that the concept of failure should be incorporated into the design of structures as a way to consider unforeseen events.

As a consequence of the 2011 earthquake and tsunami impact, the Japanese government has implemented a 2-level approach to defining tsunami risk reduction measures. This approached is based on the likelihood and severity of a tsunami (Level 1: 100 yr return period, potential for major damage; Level 2: 1,000 yr return period, potential for catastrophic impact) and the different corresponding safety performance goals for structures (MLIT, 2011). The design performance of tsunami defences should be determined according to the required performance for the Level 1 and 2 scenarios. The performance design concept was already employed in the design standard for port facilities and for coastal defences in Japan. However, the Level 2 tsunami should be considered in the next version of standards and the stability performance of defence structures should be investigated further (Takahashi et al. 2012).

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7 Industrial districts

7.1 EARTHQUAKES

7.1.1 Northridge Earthquake, USA, 17 January, 1994

7.1.1.1 Trigger characteristics

The Northridge earthquake not only affected industrial equipment, pipelines and utilities, but it also caused widespread damage in industrial buildings. Within the surface projection of the ruptured zone, where the maximum modified Mercalli intensity (MMI) measure was quantified as 9, the peak ground acceleration (PGA) attained was about 0.7g (Wald et al., 1999). Although the Northridge Earthquake did not cause surface rupture, widespread permanent ground deformations were reported (Holzer et al., 1999). More detailed information on the characteristics of the earthquake can be found in Section 2.1.3.

7.1.1.2 Impact dynamics

Damage and failure mechanisms

Considering pre-1976 tilt-up buildings, Adham et al. (1996) highlighted the failure of connections between concrete wall panels and roof diaphragms (Fig. 7.1). Other common types of failures observed in the pre-1976 buildings were large out-of-plane deformations of wall panels and collapse of suspended ceilings. Rehabilitated pre-1976 tilt-up buildings performed relatively better in terms of damage to the connection between panels and roofs, whereas collapse of suspended ceilings and cracks due to minor out-of-plane bending of panels have still been observed (Adham et al., 1996). Despite several post-1976 tilt-up buildings performing well, with minor cracking only, many buildings suffered collapse of walls due to the failure of ties between panels and beams or partial roof collapses because of inadequate shear reinforcement of columns supporting roof beams.

The Northridge earthquake also caused brittle connection failures in a large number of steel moment resisting frame (MRF) structures in industrial districts. Neither total collapse nor casualties were reported due to the connection failures of these welded steel moment resisting frame structures, but brittle damage ranging from minor cracking to completely severed columns was observed (Mahin, 1998). Miller (1998) studied the damage on these structures and emphasized the importance of workmanship-quality issues.

Fig. 7.1 Collapse of roof and wall elements of an old tilt-up building (Photo by G. Sakkestad[2])

Regarding the non-structural damage observed in industrial facilities, damage to architectural components, machinery and rack storage systems (Fig. 7.2) were common. Finally, damage to infrastructural systems such as water supply, electricity, fire proofing, emergency shutoff systems and telecommunications usually caused industrial buildings to suffer from severe post-shaking issues even where structural damage may not have been significant.
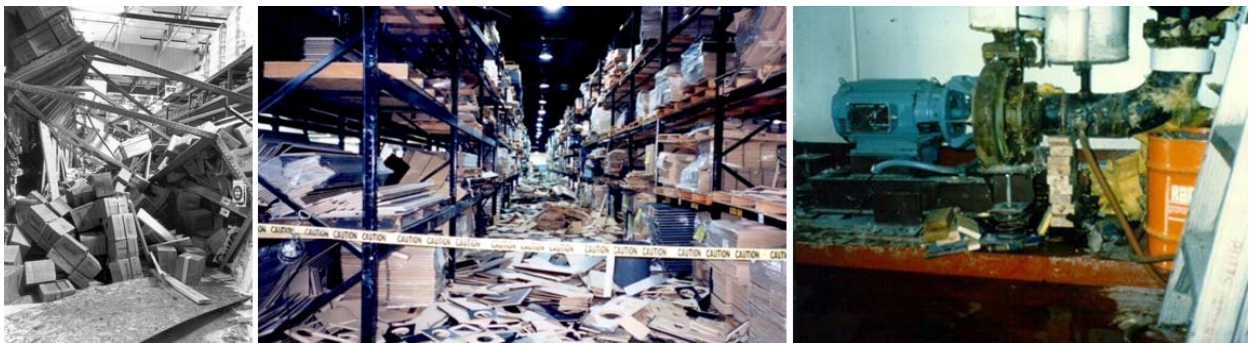
Fig. 7.2 Damage to industrial storage racks and equipment during Northridge Earthquake (Photos taken from FEMA 460 (2005) and FEMA E-74 (2011))

Protection measures and systems

Following the 1971 San Fernando earthquake, significant revisions to earthquake design requirements were made in the US. The 1976 Uniform Building Code is often specified as a "benchmark" code that introduced "modern" seismic design methods.

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[2] http://www.metroactive.com/papers/metro/10.14.99/tiltup-9941.html

The focus, however, was on providing acceptable life-safety, and the 1994 Northridge earthquake demonstrated the limitation of the building code to address post-earthquake serviceability requirements, with many commercial and industrial enterprises forced to close due to damage, much of it non-structural (Lew et al., 1994).

Due to the introduction of the aforementioned 1976 Uniform Building Code, one-storey tilt-up buildings were grouped by Adham et al. (1996) during the post-earthquake reconnaissance as pre-1976 and post-1976 buildings, as well as rehabilitated pre-1976 buildings. The seismic code revisions for tilt-up buildings were aimed mainly at improving the connection between walls and floor diaphragms and increasing the design forces for tilt-up panels and their connections (Adham et al., 1996). This earthquake thus provides insight on the efficacy of the earthquake mitigation practices that have been undertaken in Los Angeles between 1976 and 1994.

7.1.1.3 Consequences

Damage to the critical infrastructure and its components

In the industrial districts, the earthquake caused significant structural damage to one-storey tilt-up buildings as well as steel moment-resisting frames, mainly due to connection failures. Damage to the non-structural components and contents of industrial buildings was also frequent.

One-storey tilt-up buildings for industrial or commercial facilities are common in California; before the earthquake there were about 20,000 tilt-up buildings in Southern California and about 2000 of them were in the San Fernando Valley (CSSC, 1995). Nevertheless, the performance of relatively new tilt-up buildings was worse than expected and more than 350 tilt-up concrete buildings suffered moderate damage and 200 buildings were severely damaged with partial roof or exterior wall collapses (CSSC, 1995).

Socio-economic impact

The 17 January 1994 Northridge earthquake hit the San Fernando Valley and affected residential, commercial and industrialized areas of the region. Due to this earthquake, more than 12,000 structures were damaged and many residents remained homeless (Lindell and Perry, 1997). According to Tierney (1997), the total estimated cost of the damaged structures was 20 billion US$ whereas total estimated costs exceeded 40 billion US$. The earthquake caused 57 fatalities and more than 9,000 serious injuries. According to survey respondents of a sample representing the most affected areas by the earthquake, about 13% of the buildings housing businesses were reported as having severe structural damage whereas more than 50% of the business suffered different degrees of damage - almost 70% of them were non-structural.

Cascading effects

Earthquake triggered hazardous material releases at industrial facilities were observed more frequently than in previous events. Field surveys conducted in Santa Clarita and Van Nuys areas after the Northridge Earthquake showed that 39 of 134 (i.e. 29%) earthquake triggered hazardous material releases happened at industrial sites (Lindell and Perry, 1997). This observation revealed that 10% of industrial facilities storing hazardous materials suffered from this kind of incident. The Northridge earthquake showed that earthquake-initiated hazardous material accidents may occur even for cases where structural damage is slight, which indicates that chemical containment systems are more fragile than the buildings in which they are placed (Lindell and Perry, 1997).

7.1.1.4 Contributing factors

Structural connection failures and the poor performance of non-structural elements arose due to inadequate design code requirements.

7.1.2   Kocaeli earthquake, Turkey, 17 August 1999

7.1.2.1   Trigger characteristics

The Kocaeli earthquake hit an area that is considered the heart of Turkish industry, thereby causing significant damage to industrial districts and resulting in major losses in industry due to structural damage and business interruption. Details on the earthquake trigger can be found in Section 2.1.2.

7.1.2.2   Impact dynamics

Damage and failure mechanisms

Saatçioğlu et al. (2001) reported both adequately performing precast industrial facilities (Fig. 7.3a) as well as heavily damaged (Fig. 7.3b) or collapsed (Fig. 7.3c) examples with fairly similar source-to-site distances. Failures due to inadequate roof diaphragms, especially in unfinished constructions, were highlighted. For instance, Saatçioğlu et al. (2001) reported the case of two totally collapsed precast buildings under construction (Fig. 7.4a) and one finished building with diaphragm roof and side panels (Fig. 7.4c) that survived next to the others. The absence of lateral deformation constraints (i.e., roof diaphragms) caused hinging at the bottom of columns (Fig. 7.4b) in the first case whereas only distress at the beam-column connection (Fig. 7.4d) was observed in the second case.

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Fig. 7.3 Precast Industrial buildings sustained various degrees of structural damage (taken from Saatçioğlu et al., 2001)

Fig. 7.4 Precast structures in the same industrial zone under different construction stages and corresponding damage levels (taken from Saatçioğlu et al. (2001))

Other common failures observed by Saatçioğlu et al. (2001) were mid-height column failures due to partial masonry walls (Fig. 7.5a) or inadequate lateral reinforcement (Fig. 7.5b), column-beam connection failures and unseated beams from column support due to large lateral drift demand (Fig. 7.5c). Double cantilever to column connection failure (Fig. 7.6a) and collapse of double cantilever head (Fig. 7.6b) due to the failure of bolted connection to column (Fig. 7.6c) were other damage mechanisms in precast industrial buildings highlighted by Saatçioğlu et al. (2001).
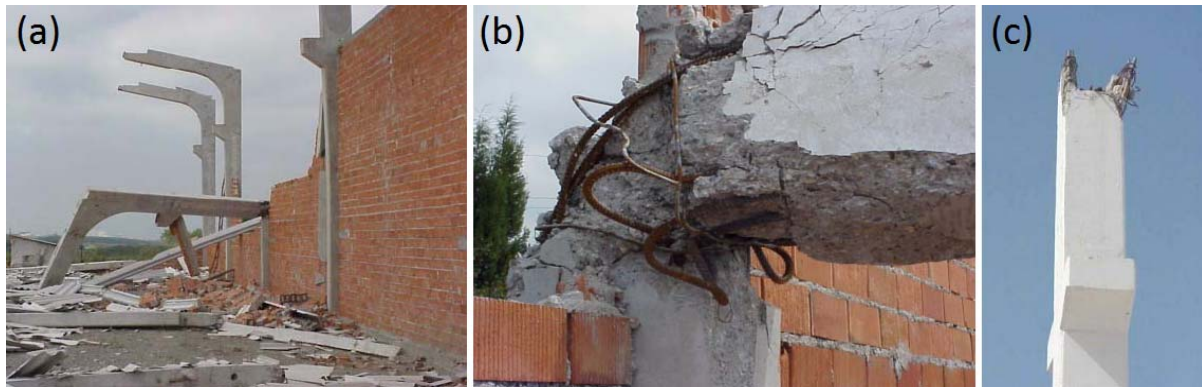
Fig. 7.5 Observed damage to precast industrial buildings (taken from Saatçioğlu et al., 2001)
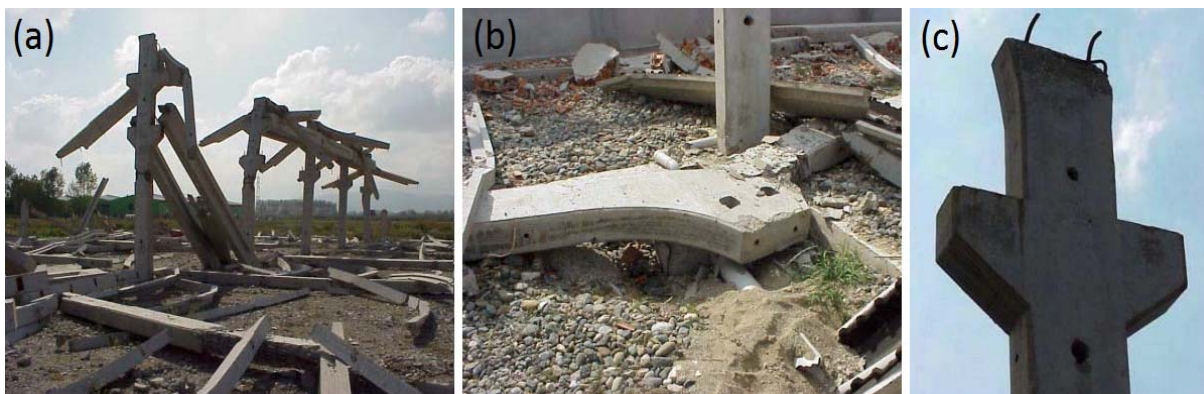
Fig. 7.6 Double cantilever to column connection failures (taken from Saatçioğlu et al., 2001)

Protection measures and systems

Pin connected precast concrete buildings constitute the majority of the Turkish industrial building stock (Şenel and Kayhan, 2010). Despite relatively better seismic performance of pin connected members compared to connections relying on friction, precast industrial facilities suffered different degrees of structural damage during 1999 Kocaeli Earthquake. One of the reasons for the relatively poor seismic performance of precast industrial buildings is the seismic design codes used in Turkey before 1998. Şenel and Kayhan (2010) state that more than 80% of existing industrial precast structures were constructed before 1998. Pre-1998 seismic codes in Turkey prescribed relatively lower seismic design forces which caused low amounts of lateral reinforcement and relatively small column cross sectional areas. The Turkish Earthquake Code published in 1998 (TEC-98, 1998) increased the seismic design force by defining a strength reduction factor for pin connected precast concrete structures equal to 5. Şenel and Kayhan (2010) pointed out that this value was still higher than the R=1.5 prescribed by Eurocode 8 (CEN, 2004).

7.1.2.3 Consequences

Damage to the critical infrastructure and its components

Various degrees of damage, either structural or non-structural, were observed in industrial structures within the region subjected to strong ground shaking. According to a survey in industrialized areas affected by the Kocaeli event, more than 50% of industrial facilities sustained structural damage (Cruz and Steinberg, 2005). Şenel and Kayhan (2010) emphasized inadequate strength, stiffness and ductility as well as insufficient connection detailing as the main reasons contributing to the vulnerabilities of precast structures, which constitute 90% of lightweight industrial facilities in Turkey.

In addition to structural damage to hazardous industrial installations, Cruz and Steinberg (2005) documented hazardous materials release incidents and damage to equipment in industrial facilities. According to their survey results, 8% of industrial facilities handling hazardous materials suffered from earthquake triggered hazardous material incidents, which was slightly less than cases observed in the 1994 Northridge earthquake.

Socio-economic impact

The 1999 Kocaeli Earthquake destroyed 18,000 structures. More than 17,000 persons were killed and over 60,000 residents were left homeless (Sucuoğlu, 2002). The number of injuries were more than 43,000 (Özerdem and Barakat, 2000).

Economic and industrial activities corresponding to more than 30% of the Turkish GNP were affected by the Kocaeli earthquake. The estimated total monetary loss was between 9 - 13 billion US$ according to the State Department for Planning of Turkey (Özerdem and Barakat, 2000). This total is mainly composed of 5 billion US$ for buildings, 2 billion US$ for industrial facilities, 1.4 billion US$ for infrastructures and additional economic losses within the period of time passed before the factories and industrial facilities returned to their pre-disaster production levels (Özerdem and Barakat, 2000).

Cascading effects

Some critical industrial facilities suffered from earthquake-triggered fires which continued for many days and threatened to cascade onto other facilities. These fires mainly affected large industrial facilities, such as the TUPRAS oil refinery, rather than common industrial precast structures. The impact of the Kocaeli earthquake on the TUPRAS refinery is discussed in detail in Section 2.1.2.

7.1.2.4 Contributing factors

Olgiati et al. (2011) indicate that the fundamental period of a typical one-story reinforced concrete precast structure is between 0.8s and 1.4s according to a study among buildings in Italy, Greece, Slovenia and Turkey. Regarding the response

spectra of accelerograms recorded within a 15 km source-to-site distance (Sucuoğlu, 2002), fault-parallel components reached up to 1.5g within the aforementioned period range whereas fault-normal components ranged between 0.2g and 0.9g. These high amplitudes of spectral accelerations (and associated displacements) contributed to the high damage observed in industrial facilities near to the source.

7.1.3   L'Aquila Earthquake, Italy, 6 April 2009

7.1.3.1   Trigger characteristics

With the epicentre of the earthquake being less than 10 km from the urban centre of L'Aquila significant damage to structures occurred including industrial districts in the neighbouring areas. Augenti and Parisi (2010) report a maximum level of X on the Mercalli-Cancani-Sieberg macroseismic scale whereas Global Risk Miyamoto (2009) report and MMI of VI-VII. Detailed information on the L'Aquila earthquake characteristics is provided in Section 5.1.1.

Although the L'Aquila earthquake was moderate in terms of moment magnitude, the damage observed was widespread. Comparison of the acceleration spectrum obtained within 10 km source-to-site distance with the spectral values taken from the Italian Building Code (NTC08, 2008) shows that for periods smaller than 1 second, the observed spectral acceleration was in the range of values for a return period of 2,475 years (Masi et al., 2011), rather than 475 years as is currently used in design.

7.1.3.2   Impact dynamics

Damage and failure mechanisms

Bazzano, Pile and Sassa are the industrial areas near to the city of L'Aquila where precast concrete panel buildings and precast walls connected either to reinforced concrete frames or steel frames are the most common structural types (Grimaz et al., 2010). Toniolo and Colombo (2012) indicate that the majority of the precast structures in this region was used as one-storey industrial facilities whereas few of them were two to three storey commercial buildings. According to field surveys, Grimaz et al. (2010) report common structural damage, such as unseated beams due to relative movement of the beam and column corbel, column-beam connection failures and column shear failures. They also identified collapsed steel silos which collided with the adjacent precast structure or fell down due to heavy damage at their base.

Toniolo and Colombo (2012) highlighted the relatively good performance of new precast buildings in terms of the main structural integrity as well as unsatisfactory seismic performance of panel-structure connections. Nevertheless, Toniolo and Colombo (2012) observed some structural damage, such as fallen beams or roofing elements from their bearings (Fig. 7.7a), fracture of column pocket supports (Fig. 7.7b) and buckling of longitudinal reinforcement at columns mid-height (Fig. 7.7c).
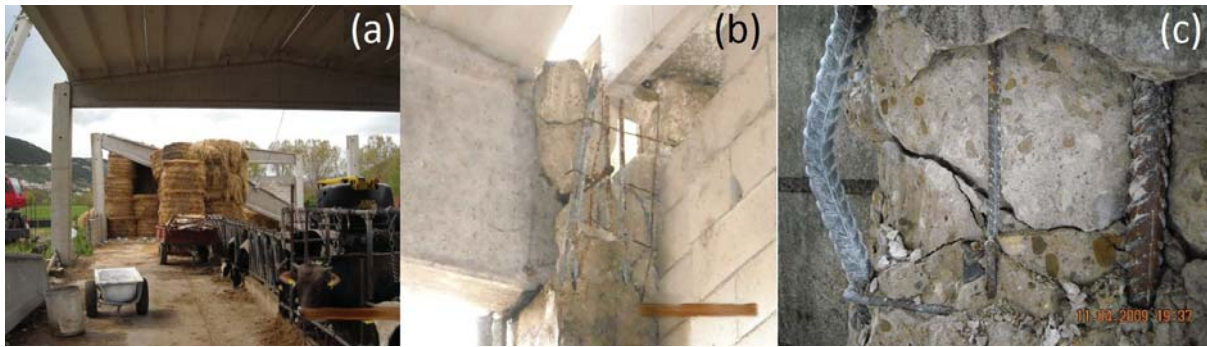
Fig. 7.7 Damage to structural elements observed by Toniolo and Colombo (2012)

Damage to non-structural elements as well as equipment and machinery were observed in industrial buildings. Even for new industrial buildings either collapse of non-load bearing precast panels, roofing elements or unreinforced masonry infill walls were reported. Global Risk Miyamoto (2009) explained this kind of damage as insufficient anchorage or improper connection detailing of wall elements and roofing systems between the frames of the structure. Toniolo and Colombo (2012) also attributed the damage to connections and fastenings (e.g. channel bars), shown in Fig. 7.8, which were not designed against seismic forces in the tangential transverse direction.

Fig. 7.8 Damage to connections and fastenings (Toniolo and Colombo, 2012)

Di Sarno et al. (2011) indicated that the ratio of the vertical to horizontal component of the L'Aquila earthquake was relatively high especially for sites with a source-to-site distance of less than 10 km. Furthermore, they showed that the time between the occurrence of peaks in the vertical and horizontal acceleration time histories is relatively short. As a consequence, the relatively high damage to non-structural components (e.g. collapse of wall panels or unanchored equipment) of industrial facilities close to the epicentre may have been amplified because of these particular characteristics of the earthquake (see Fig. 7.9). Toniolo and Colombo (2012) note that the collapse of panels would not have occurred if the planned concrete topping had been added to this uncompleted construction before the earthquake.

Fig. 7.9 Collapse of precast wall panels (Di Sarno et al., 2011)

Protection measures and systems

First attempts to include provisions for the design of reinforced concrete precast structures in seismic zones in Italy date back to 1987 and 1996, but an appropriate design code including a specific chapter about precast structures was only introduced in 2003 (OPCM 3274) and in 2008 (NTC08, 2008). There are a very limited number of precast buildings in Italy designed in accordance with these latter regulations. The area of L'Aquila has been classified as a seismic zone since 1915, and a number of changes (increases) to the lateral load force applied in the design of buildings have been made over the years. For what concerns precast structures, as mentioned above, the main design modifications were applied in the late 80's and early 90's. Nevertheless the allowable tension stress design method was still applied, but p-delta effects were included, shear reinforcement was explicitly designed and the connections were dowel as opposed to friction.

7.1.3.3 Consequences

Damage to the critical infrastructure and its components

This M_w 6.3 earthquake caused significant damage in the city centre of L'Aquila as well as in the surrounding residential and industrial areas. Augenti and Parisi (2010) reported that more than 60,000 buildings were damaged to some degree. The damage of reinforced concrete structures mainly originated from a lack of proper reinforcement detailing, irregularities either in plan or elevation and formation of soft storey behaviour due to lack of infill walls.

Relatively more heavy damage was observed in the southern part of L'Aquila than in the northern part which can not be explained easily only with source to site distance. Assuming similar structural characteristics for both of these regions, damage is probably amplified by site effects combined with rupture directivity or near-field pulse-like effects (Masi et al., 2011). Nevertheless, the observed damage was highly correlated with the age and structural type.

Socio-economic impact

The damage to residential areas and industrial districts was considerable. The L'Aquila earthquake caused more than 300 fatalities, 1500 injuries and about 29,000 homeless residents (Augenti and Parisi, 2010). The total cost of the damage, including indirect losses, was estimated to exceed 16 billion US$ (Global Risk Miyamoto, 2009).

Cascading effects

Generally, past earthquakes have highlighted the vulnerability of industrial facilities that contain chemical substances due to damage of deficient (according to today's standards) reinforced concrete or fragile precast structures widely common in Europe. However, no environmental impact from chemical substances was reported for the L'Aquila earthquake.

7.1.3.4 *Contributing factors*

Toniolo and Colombo (2012) highlighted the inadequacy/deficiency of the current design philosophy of frame systems with precast wall panels where the design is based on load resisting bare frame systems with additional masses coming from the existence of precast panel walls. The observations after the L'Aquila earthquake showed that the damage to frame structure with peripheral panel walls behaved primarily (until the collapse of connections and panel walls) like a dual wall-frame system with higher stiffness which caused higher lateral forces and damage (Toniolo and Colombo, 2012).

7.1.4 **Christchurch Earthquake, Australia, 22 February 2011**

7.1.4.1 *Trigger characteristics*

New Zealand's second most populated city, Christchurch, was hit by a M_w 6.3 earthquake on 22 February 2011 almost 6 months after the M_w 7.1 Darfield earthquake that occurred about 30 km to the west of Christchurch (Bradley and Cubrinovski, 2011). Kaiser et al. (2012) pointed out that the Christchurch earthquake occurred on a blind northeast-southwest-striking fault with the epicenter 6 km away from the city centre. The focal depth of the earthquake was 5 km (Kam and Pampanin, 2011). The 12x6 km^2 rupture of the oblique-reverse fault did not cause a surface slip (the maximum slip was about 3.6 m at 3.5 km depth) but created peak ground accelerations of up to 0.7g in the city centre and 2.2g near the epicentre (Kaiser et al., 2012). The rupture duration was 4 seconds and was oriented towards Christchurch which contributed to the relatively higher intensities experienced in the city (Holden, 2011).

Reyners (2011) and Gledhill et al. (2011) have both discussed the known activity of the Alpine Fault system located along the north-western part of South Island which ruptures in every 200 - 300 years causing earthquakes with magnitudes larger than

7.5. Coupled with the Alpine fault, the Southern Alps and other seismically active areas (similar to the one that caused 2010 Darfield and 2011 Christchurch earthquakes) are prone to cause magnitude >6 earthquakes (Gledhill et al., 2011).

7.1.4.2 Impact dynamics

Damage and failure mechanisms

The majority of the commercial buildings in Christchurch are reinforced concrete frame structures located in the central business area and tilt-up industrial buildings. Marshall and Gould (2012) classified low-rise industrial buildings in Christchurch into three types: structures composed of load bearing tilt-up precast concrete panels with steel roof framing, pre-engineered steel frames with precast concrete cladding panels, and pre-engineered steel frames with a light gauge metal or insulated metal panel cladding.

Kaiser et al. (2012) pointed out that the tilt-up panel construction methodology widely used in industrial buildings suffered structural precast panel element connection failures. Also steel rack storage systems were heavily damaged in industrial buildings because of the extreme shaking intensity near the epicentre (Kaiser et al., 2012). Nevertheless, liquefaction was widespread and also affected industrial buildings.

Concrete wall panels of tilt-up structures cracked either diagonally (Fig. 7.10a) because of high shear stresses or horizontally (Fig. 7.10b) because of excessive out-of-plane deformation.
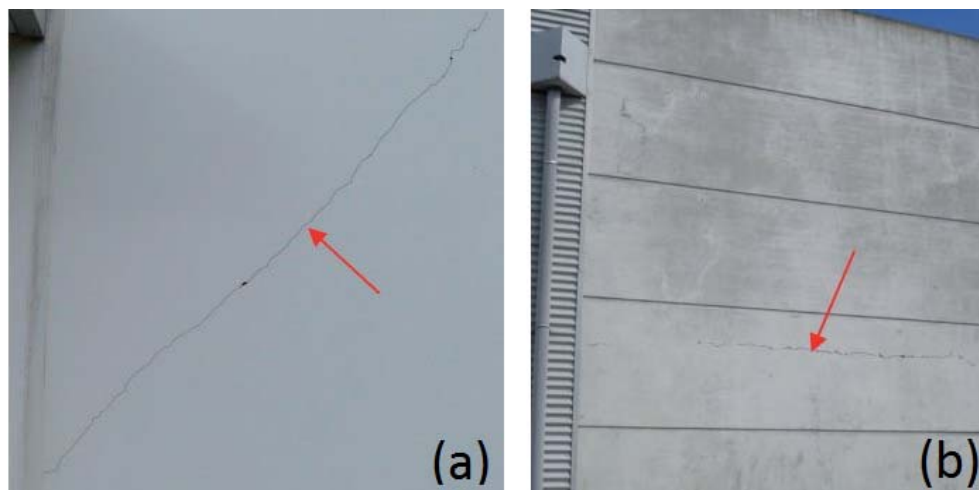
Fig. 7.10 Minor cracking in concrete panels (taken from Henry and Ingham (2011))

Henry and Ingham (2011) indicated that tilt-up panels are usually connected to the structural system only at the foundation and roof level. They concluded that this type of restraint cannot prevent the system exhibiting relatively large out-of-plane deformations (Fig. 7.11) which cause horizontally cracked sections. Industrial structures with load-bearing tilt-up concrete panels suffered mainly either the

cracking of panels due to poor (light) reinforcing or poor connection design (Marshall and Gould, 2012).

Fig. 7.11 Out-of-plane buckling of concrete wall panels (taken from Henry and Ingham (2011))

Regarding the joints between structural components, Henry and Ingham (2011) indicated the relatively poor performance of vertical joints between wall panels which is much more evident in the case of architectural irregularities such as building sections with different structural heights. Marshall and Gould (2012) also highlighted the poor performance of connections between the steel frame resisting gravity loads and concrete panels resisting lateral loads which caused wall panels to detach from the structure and collapse (Fig. 7.12).

Fig. 7.12 Panel collapse due to steel frame connection failure (taken from Marshall and Gould (2012))

Kaiser et al. (2012) showed that liquefaction was widespread and caused damage to more than 15,000 residential houses, infrastructural systems, high-rise buildings and bridges. Cubrinovski et al. (2011) described damage due to punching settlement and

differential settlement that was experienced by industrial buildings, especially those with shallow foundations. For some other cases, liquefaction and large ground deformation resulted in damage to critical equipment because of significantly exceeded levelness tolerance (Marshall and Gould, 2012).

Protection measures and systems

Tilt-up industrial buildings exhibited limited damage during the Christchurch earthquake considering the earthquake shaking intensities. Relatively short vibration periods of tilt-up systems, the short shaking duration and the New Zealand concrete design standards that ensure that tilt-up structures are designed to resist seismic loads within the elastic limits or with a very limited inelastic ductile response are some of the major reasons for fairly good seismic performance of this type of system (Henry and Ingham, 2011).

7.1.4.3   Consequences

Damage to the critical infrastructure and its components

The maximum intensity in terms of MMI was quantified as IX by Kam and Pampanin (2011). Coupled with basin effects, directivity effects and shallowness, the ground shaking intensity was higher than usual. For instance, Kaiser et al. (2012) showed that the response spectra of accelerograms recorded within the business centre were generally higher than the New Zealand design response spectrum for a return period of 2500 years for periods larger than 0.8 seconds. Assuming design spectra for a 500-year return period specified in NZS1170:2004 (2004) as the design level, the expected moderate damage (life-safety performance level) was observed in buildings constructed after 1990s whereas pre-1980 (especially pre-1970) structures generally suffered from brittle failures and heavy damage (Kam and Pampanin, 2011).

Observations show that most of the post-1990 industrial buildings did not suffer from severe or irreparable structural damage but damage to non-structural components and architectural members was extensive (Kaiser et al., 2012) which eventually yielded unsatisfactory performance and disruption in production.

Socio-economic impact

The Christchurch earthquake caused more than 180 deaths and heavily damaged or totally collapsed buildings in the city centre. Kaiser et al. (2012) emphasized that more than 900 buildings mostly located in the central business district and about 10,000 residential buildings suffered heavy damage and had to be demolished.

Kaiser et al. (2012) described the Christchurch earthquake as the most destructive earthquake in New Zealand's history in terms of physical damage and repair cost. Considering Christchurch's 15% contribution to New Zealand's GDP (Reyners,

2011), the total economic loss will increase further until the production dynamics return to their pre-disaster levels.

Cascading effects

The destruction in industrial districts led to severe economic losses for businesses, due to both damage to buildings and the loss of production. No releases of hazardous materials potentially causing fires, explosion or environmental damage were reported for the Christchurch earthquake.

7.1.4.4 Contributing factors

The vertical acceleration levels were unexpectedly large combined with the relatively high horizontal accelerations. This phenomenon also increased the non-structural damage during the earthquake. For instance, Marshall and Gould (2012) reported movement of equipment weighing 110 kN of about 1.2 m.

Kam et al. (2011) indicated that post-earthquake rapid screening and assessment procedures alone may not capture all weak or risky structures. For instance, some of the collapses of commercial buildings that occurred during Christchurch Earthquake were perhaps not identified as being at risk by the rapid evaluation that was applied after the Darfield Earthquake that hit the same region 6 months before.

7.1.5 Emilia Romagna Earthquakes, Italy, 20 and 29 May 2012

7.1.5.1 Trigger characteristics

On 20 and 29 May 2012, the Emilia Romagna region of Italy was hit by two earthquakes with M_w 6.1 and 5.9, respectively (Pondrelli et al., 2012). A seismic sequence was observed in the Emilia Romagna region, one of the most industrialized areas of Italy, during the hours before and after both events. According to Bignami et al. (2012), the hypocentral depth of these events was 5.1 km and 4.2 km, respectively. Thrust faulting is assigned for both events with small strike-slip mechanism for the latter by Serpelloni et al. (2012). Both ruptures occurred over blind faults about 40 km north of Bologna, Italy (Bignami et al., 2012). Surface faulting was not observed, however, secondary ground deformations such as liquefaction was widespread (Pizzi and Scisciani, 2012).

The maximum horizontal PGA of the 20 May event was about 260 cm/s^2 from a station with 17 km epicentral distance, whereas for the 29 May event it was 290 cm/s^2 which was recorded 2 km away from the epicentre (Bournas et al., 2013). Bournas et al. (2013) stated that the corresponding maximum vertical PGA values were 303 cm/s^2 and 900 cm/s^2, respectively. Maximum reported MMI values were V and VIII for the 20 May and 29 May event, respectively (USGS, 2014).

7.1.5.2 Impact dynamics

Damage and failure mechanisms

According to field surveys, Liberatore et al. (2013) and others (e.g., Magliulo et al., 2013; Bournas et al., 2013; etc.) classified the major structural damage as hinging of column base (Fig. 7.13a), fracture of column-beam connections (Fig. 7.13b), short-column failure (Fig. 7.14b) and unseated beam (Fig. 7.14a) or roof elements whereas common non-structural damages such as cladding panel wall failures and steel rack storage failures. Cladding panel wall failures and column damages were observed in 50%, whereas beam failures were observed in about 30% of the investigated buildings that were mostly designed according to the 1987 code (Liberatore et al., 2013).

Fig. 7.13 Observed damage to industrial buildings (taken from Liberatore et al. (2013))

The Emilia Romagna events caused liquefaction and ground cracks within regions close to epicentre. Papathanassiou et al. (2012) verified these ground deformations with field surveys and reported consequent damage to industrial buildings whereas Liberatore et al. (2013) did not observe evidence of structural damage in these structures due to foundation settlement. It is likely that liquefaction was not a major contributor to the damage in this earthquake.

Protection measures and systems

Current building codes emphasize a minimum longitudinal reinforcement ratio, maximum spacing of lateral reinforcement, thickness/depth ratio of beams, ductile-strong wall frame connections etc. which were either not prescribed or less than the current minimum value in the 1987 code which contained the first specific regulations for precast structures in Italy (Magliulo et al., 2013). Relatively good structural performance of buildings designed according to the current codes prevented more fatalities and reduced monetary losses whereas lack of strengthening of old buildings (which had no seismic design provisions at all according to the current regulations increased the losses.

7.1.5.3 Consequences

Damage to the critical infrastructure and its components

Similar to the L'Aquila event in 2009, precast wall cladding panels suffered heavy damage due to the Emilia Romagna sequence. On the other hand, the Emilia Romagna events also caused severe structural damage to industrial buildings. A major reason for this was the seismic classification of the region, which was not considered until 2003 (Liberatore et al., 2013). According to Bournas et al. (2013), 75% of the total industrial building stock in this region has not been designed to resist seismic forces which caused extensive structural damage and loss of human lives.

According to field observations, Magliulo et al. (2013) concluded that the damage to precast structures in the Emilia-Romagna region was heavier than those in Turkey after the Kocaeli 1999 earthquake despite the relatively large ground shaking experienced in Kocaeli. The main reason for this observation is given as the poor performance of roof-beam and beam-column connections relying on friction (Fig. 7.14c) with respect to the doweled connections used in Turkey (Magliulo et al., 2013). It was noted, however, that catastrophic failures of precast structures with pinned beam-column connections were observed in both Emilia Romagna and Kocaeli earthquakes because of inadequacy of connection or column top section. These kind of catastrophic failures of horizontal elements caused significant damage to the structure as well as the equipment and goods stored in these facilities.

Damage to the equipment and stored goods as well as the storing facilities such as steel rack storages can cause loss of money and even human lives. Among these, Liberatore et al. (2013) discussed the vulnerability of steel rack storages and showed that these storing facilities can be independent of structure, partly integrated with the structure as in the case of supporting roof members or the structure itself (See Fig. 7.13c). Liberatore et al. (2013) concluded that the primary reasons of the partial collapse on 20 May included highly flexible structural systems without adequate bracing and heavy masses, inducing high inertial forces.
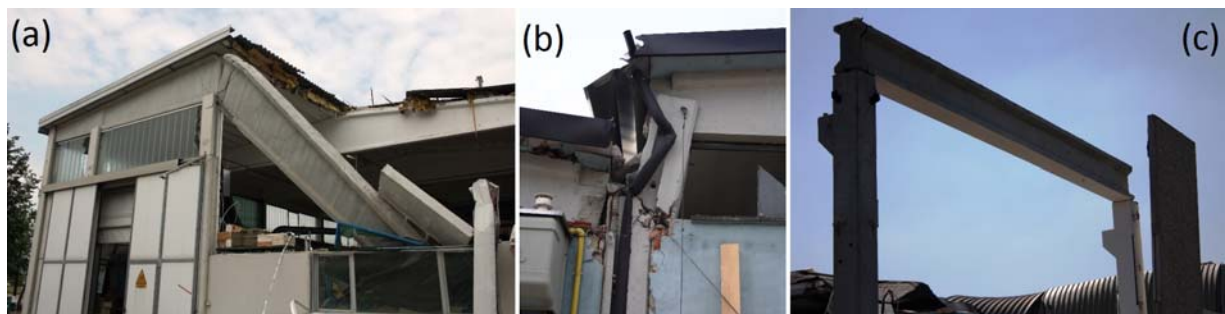

Fig. 7.14 Observed damage to beam-column connections (Source: Roberto Nascimbene, Eucentre)

Socio-economic impact

The 20 May 2012 event caused 7 fatalities whereas 20 persons were killed by the event on 29 May. A total of 400 injuries were reported and a total number of homeless people of about 15,000 (Magliulo et al., 2013). During the sequence, buildings collapsed in the municipalities of San Felice sul Panàro and Finale Emilia, where severe damage to old structures and industrial buildings occurred (Liberatore et al., 2013). For instance, Liberatore et al. (2013) reported severe structural damage to approximately 500 factories in the Modena district. The total direct loss of properties is estimated as 2 billion Euros which increases to 5 billion Euros when indirect losses such as business interruption are included (Liberatore et al., 2013). Besides monetary losses, Liberatore et al. (2013) indicated that up to 7,000 employees lost their jobs because of direct or indirect effects of this disaster.

Cascading effects

The damage and destruction to industry in the area impacted by the Emilia Romagna earthquake had severe economic repercussions on affected companies.

7.1.5.4 Contributing factors

The damage caused by the 20 May event increased the fragility of the industrial structures in the region, and led to increased damage and collapse in the 29 May event.

In addition to relatively poor seismic design of existing precast structures, the ground motion intensity of the Emilia Romagna events was relatively high, which exacerbated the damage. For instance, Magliulo et al. (2013) showed that the E-W component of the 20 May event recorded by a station 17 km away from the epicentre on stiff soil was larger than the design spectra of 475 years for periods between 0.1 seconds and 2 seconds. The N-S component of the same record is even higher than the 2475-year design spectrum for period values ranges from 0.6 seconds to 2 seconds. Magliulo et al. (2013) stated that the fundamental elastic period of precast structures designed for Italian low-to-high seismic zones is between 0.54 seconds and 1.45 seconds for bare frames and 0.09 seconds and 0.40 seconds for structures with panels.

Liberatore et al. (2013) and Bournas et al. (2013) raised the problem of the complexity of safety evaluations of industrial buildings. This process is extremely time consuming and will interrupt the process of recommencing normal business operations, thereby causing monetary losses even for structurally undamaged facilities.

7.2   LESSONS LEARNED FROM EARTHQUAKES

7.2.1   Systems weaknesses and critical components

Earthquakes have caused widespread damage that included many tilt-up and steel-frame industrial buildings. In the case of tilt-up buildings, connections between floor diaphragms and wall panels, inadequate steel reinforcement of members and large out-of-plane deformation demands of wall panels reduced the seismic performance of this kind of industrial facilities. These weaknesses usually caused partial roof collapse or suspended ceiling failures. Poor seismic performance of inadequately designed panel-structure connections was the primary reason for wall damage whereas pounding of roof and column members to the cladding wall panels also caused damage (Magliulo et al., 2013). Based on experiences from the Christchurch earthquake, Henry and Ingham (2011) indicate that design considerations related to the strength hierarchy of structural components as well as the strength ductility and anchorage of the connections should be reviewed to prevent these types of buildings from impacts due to future seismic loading.

Compared to the damage to tilt-up construction, steel-frame buildings performed relatively better at first glance during the Northridge earthquake. However, detailed investigations revealed brittle fractures in welded beam-column connections which were usually hidden with architectural cladding elements (Mahin, 1998). In Christchurch the damage to steel industrial buildings was identified as minor, with a need for replacing or re-tightening of damaged bracing elements (Canterbury Earthquakes Royal Commission, 2011).

During the Kocaeli earthquake precast industrial facilities built before 1998 were vulnerable to seismic loading due to the relatively low design forces prescribed by the code and insufficient reinforcement detailing. Furthermore, buildings that did not even satisfy the regulations applicable during their period of construction increased the number of total collapse cases. In addition, inadequate precast member connections, irregularities causing short-column behaviour, unsettlement of beams due to excessive lateral deformations and inadequate diaphragm floors/roofs increased the vulnerability of industrial building stock. In this context, Saatçioğlu et al. (2001) raise the issue of the temporarily increased vulnerability of industrial precast buildings under construction and lacking floor/roof diagrams that requires further investigation.

In the industrial districts affected by the L'Aquila earthquake structural weaknesses such as vertical irregularities prone to causing soft-storey behaviour or significant movement of the beam and column support were identified. This could lead to total collapse in the case of a stronger earthquake (Global Risk Miyamoto, 2009). Global Risk Miyamoto (2009) further lists the reasons for failure as heavy roof systems, which increase the inertial forces, and improper design of beam-column connections against seismic forces or shrinkage stresses for precast industrial buildings. Neither the Italian code (NTC08, 2008) nor Eurocode 8 (CEN, 2004) contain specific design considerations for connections. Damage to and failure of columns observed during

the Emilia Romagna earthquakes were due to low longitudinal reinforcement ratio, large spacing of stirrups, weak column pocket sockets and irregularities such as adjacent stiff members or partly infilled walls. Regarding short-column formation, one observation of Liberatore et al. (2013) deserves special attention. The sawtooth type of roofing with inclined beam system is prone to short column formation due to a sudden change of stiffness at lower beam-column connection (Fig. 7.15a) whereas knee portals illustrated in Fig. 7.15b do not have such a weakness (Liberatore et al., 2013). Common damage to beam and roof elements occurred because of either unseating of these members or pounding with adjacent structural elements. Additionally, an insufficient diaphragm effect and individual column footings are other factors that contributed to the structural damage.
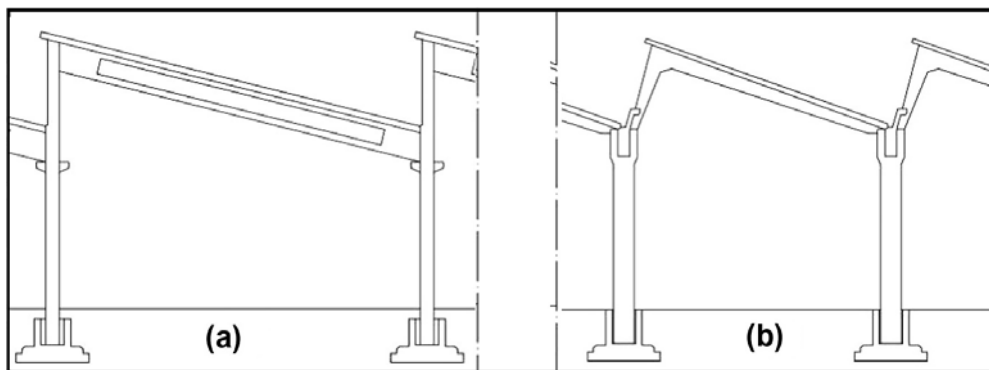
Fig. 7.15 Inclined beam (a) and knee portal (b) sawtooth roofs (Liberatore et al., 2013)

Earthquake impacts on non-structural components, e.g. heavy damage to cladding panels, as well as damage to machinery and equipment due to a lack of proper immobilization are common in industrial facilities, highlighting the importance of proper seismic anchoring of non-structural elements. In addition, there is a need for considering a serviceability limit state in the design of the system, which is not just a building, but a structure with non-structural components and content. Damage to non-structural elements can add significantly to the overall monetary loss.

In the case of industrial installations containing hazardous materials, Tang (2000) highlights the weakening effects of corrosion, chemical attack and low maintenance on originally earthquake-resistant industrial facilities and their components.

7.2.2 Potential for propagation

Even without significant structural damage, cascading effects can be observed in industrial facilities. Earthquakes have highlighted the vulnerability of industrial facilities that contain chemical substances, and hazardous material incidents involving fires or explosions can cascade onto other infrastructures. In addition, it was observed that chemical containment systems can be more fragile than the buildings in which they are placed, and thus the performance of these components should also be considered when designing industrial facilities that house or process hazardous materials. Preparedness of the industrial facilities to this type of accidents

can be improved significantly by retrofitting activities, introduction of new seismic design standards, better stabilization of storage facilities and developing emergency management strategies (Cruz and Steinberg, 2005).

Where sequences of earthquakes are likely, it should be considered that industrial structures already damaged in the preceding earthquake will be more fragile and prone to damage in the subsequent event. Simple but reliable procedures for rapid vulnerability assessment of precast structural systems might help reduce the impact of the following event (Liberatore et al., 2013; Bournas et al., 2013).

Cascading effects to a region's or country's economy are also possible where strong earthquakes affect extended areas that are heavily industrialised. These effects can be caused by structural damage and production downtimes, as well as via a ripple effect due to supply-chain interruption in otherwise undamaged facilities.

7.2.3 Consequence severity and extent

The socio-economic impact of earthquakes on industrial districts can be severe and stems from structural and non-structural damage, as well as from damage to raw materials or products to a potentially high number of individual businesses over a large area. In addition, industrial facilities might not able to get back to business immediately after the event due to damage to utilities that provide services to these facilities (e.g. water, electricity) or disruption of other infrastructures (e.g. access roads, communication systems).

7.2.4 Protection measures and systems

The implementation of protection measures and systems requires an understanding of the type and severity of the earthquake forces impacting industrial districts. Past earthquakes have shown a need for a better quantification of the seismic hazard, as well as the factors that contribute to high ground shaking intensities (Kaiser et al., 2012). Also soil-structure interactions have to be better understood to prevent liquefaction-induced damage in structures that otherwise performed well during the seismic loading (Kam et al., 2011). Hazard assessment and the corresponding codified seismicity in terms of both horizontal and vertical design forces should be kept up to date.

Most facilities affected by the Kocaeli earthquake were very close or within the surface projection of the fault. Tang (2000) notes the influence of geological factors on damage to industrial facilities and raises the issue of site-specific vulnerability. Özerdem and Barakat (2000) indicate poor city planning strategies which did not consider the soft soil deposits or the proximity to faults as a major factor for increasing the vulnerability of industry. Consequently, the primary action of public authorities was to introduce new land-use regulations and compulsory insurance for earthquake losses. However, in this context it should be noted that insurance is a mechanism for transferring risk but not for reducing it.

Much of the damage and destruction in past earthquakes can be attributed to insufficient or inadequate design requirements. Updating of existing codes or the development and implementation of new codes following high-impact earthquake might be indicated. Following the Northridge earthquake, technical documents and guidelines for construction, metallurgy, welding, quality assurance and in-process visual inspection for welded steel moment resisting frame structures were prepared (Miller, 1998). With these documents, the connection detailing prescribed by the Uniform Building Code for seismic applications during the period 1985-1994 was changed. Eventually, recommended seismic design criteria for new steel moment-frame buildings was published (FEMA 350, 2000). The seismic performance of precast structures during the Kocaeli Earthquake highlighted the inadequacy of column strength and stiffness. In 2007, the Turkish Earthquake Code (TEC-07, 2007) was updated to a strength reduction factor of 3 (reduced from the value 5 used in 1998) which is more compatible with Eurocode 8 (CEN, 2004) provisions.

Experience with the recent earthquakes in Italy and subsequent hazard analyses point towards higher seismic levels than those used for the design of most structures in central Italy, making industrial districts vulnerable. In addition to implementing retrofitting measures where possible and financially viable, current design approaches should be revised to consider inherent ground-motion uncertainties more rigorously. For instance, near-fault and directivity effects which amplify damage to industrial buildings and corresponding facilities, as well as proper vertical-to-horizontal spectral ratios should be addressed by the new generation of design codes.

For new structures, the need for a better seismic design approach for panel-structure connections is evident. Eurocode 8 (CEN, 2004) does not include specific seismic design considerations for connections. Toniolo and Colombo (2012) suggested either designing new connections that allow large frame displacements under seismic forces or designing the whole structure as dual wall-frame systems and corresponding fixed connections. Liberatore et al. (2013) suggests connected sleeve footings, thicker floor diaphragms, strong and ductile column-beam connections with rubber interfaces to prevent pounding failures of concrete elements.

Liquefaction-induced damage indicates that more sophisticated foundation design approaches should be investigated (Kam et al., 2011). For instance, buildings with pile foundations are reported as performing relatively well in terms of differential settlement or tilt caused by ground deformations (Kam and Pampanin, 2011).

The Northridge earthquake highlighted that design codes should also consider the serviceability limit states, and not just life-safety due to structural damage, because non-structural damage can be even more important than structural damage for business interruption and cascading effects (such as hazardous materials releases). Regarding post-disaster mitigation strategies for businesses, Tierney (1997) promotes enhancing the resilience of lifelines and infrastructure systems with respect to the updated seismic hazard information, and increasing the flow of goods and services to increase the resistance of industrial facilities to the impacts of disaster.

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7.3   FLOODS

7.3.1   Thai Floods, Thailand, 2011

7.3.1.1   *Trigger characteristics*

In 2011, the maximum flood ever recorded in Thailand struck the central region of Thailand. The combination of strong summer monsoon rains with 5 typhoons (Haima, Nock-Ten, Nesat, Haitang, Nalgae) occurring between June and October caused a massive flood with a volume of about 15 billion m^3 (Komori et al., 2012). The total rainfall amount in 2011 was about 40% more than the average of the years 1982-2002 (Komori et al., 2012) and it is the highest one in the last 61 years in Thailand (Gale and Sounders, 2013). Moreover, 5 typhoons in one year is significantly more than the yearly average which is given as 1.5 (Komori et al., 2012)). As a result, the flood triggered at the end of July spread to northern, north-eastern and central Thailand along the Mekong and Chao Phraya river basins, reached the neighbourhoods of Bangkok in October and remained in some regions until January 2012 (Rerngnirunsathit, 2012). The water gates and levees collapsed because of the overflowing and uncontrollable excessive amount of water that caused damage to regions near to the Chao Phraya River basin (Komori et al., 2012).

Gale and Sounders (2013) investigated the historical data, satellite-derived river flow rates as well as heavy rainfall frequency in Thailand and concluded that the return period of events similar to the 2011 flood in the Chao Phraya River basin is about 10-20 years if the current mitigation system against floods in Thailand is not improved further.

7.3.1.2   *Impact dynamics*

Damage and failure mechanisms

Damage to the industrial sector was extensive and severe due to the inundation height and the duration of the flood. Van Oldenborgh et al. (2012) stated that some industrial regions experienced more than 2.5 m inundation level for 2 months (see Fig. 7.16). The water entered the industrial sites when embankments were overtopped or breached, or via discontinuities in retaining walls.

Generally, structural damage to the typically one-storey main buildings was very limited as the speed of the arriving floodwaters was low (Aon Benfield, 2012). The bulk of the damage was caused by the intrusion of water that remained in the facilities for weeks. This resulted in the rotting of wooden walls and mold, irreparable damage to high-end laboratory equipment and precision parts, and damage to raw materials and products due to high-humidity levels caused by the stagnant flood waters. Damage to clean room environments was caused by the heavily contaminated water (Aon Benfield, 2012).

Fig. 7.16 Inundation level at industrial estates near northern Bangkok[3]

Protection measures and systems

Thailand is in a tropical region where typically monsoon rainfalls and typhoons occur especially from May to October. Since the country is prone to excessive amounts of rainfall every year, Thailand has flood control structures like levees and barriers next to the river banks, drainage canals, pump stations and dams used for flood mitigation purposes. The Bhumibol and Sirikit dams stored approximately 10 billion m^3 of water which reduced the level of damage considerably (Komori et al., 2012).

In addition to dam reservoirs, there were water pumping stations near Bangkok with a capacity of more than 1000 m^3 per second. Unfortunately, the majority of this capacity was designed to discharge the water into the Chao Phraya River of whose water level was already higher than the parapet height over three weeks (Komori et al., 2012). Besides, the levees placed on both sides of the river had been damaged because of the flood (Rakwatin et al., 2013).

Regarding the flood mitigation systems, the extensive drainage system of the country performed relatively poorly because of high tides in the Gulf of Thailand as well as congestion by waste and man-made structures (Rakwatin et al, 2013). In their paper, Rakwatin et al. (2013) reported that high tides raised the level of Chao Phraya River higher than the floodwall of the city.

Embankments around factories could not keep the floodwaters out. On the one hand, this was due to the poor condition of the embankments that resulted in a low resistance against breaching by the water. On the other hand, embankments were below the highest water mark observed and were simply overtopped (Aon Benfield, 2012).

Some companies moved machinery and products to elevated parts of the factory to escape inundation. This saved the machinery; nonetheless, products were lost due to the high humidity and subsequent formation of mold (Aon Benfield, 202).

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[3] http://www.bbc.co.uk/news/world-asia-pacific-15335721

7.3.1.3 Consequences

Damage to the critical infrastructure and its components

Limited structural damage was observed, with the majority of the damage to industrial facilities being caused by the prolonged high levels of flood water which caused damage to contents and non-structural elements.

Socio-economic impact

The 2011 Thailand Flood was the first that caused inundation in the center of Bangkok. Disaster damage was the most severe one in Thailand's history and it was the fourth at a global scale (Koontanakulvong, 2012). In total, 13.6 million people were affected by the flood which caused 815 fatalities and severe damage over 20,000 km^2 (Rerngnirunsathit, 2012). According to the 2011 Thai Flood report by The World Bank (2012), the total cost of flooding damage and economic losses was estimated as more than 45 Billion US$.

Seven industrial parks which were home to 804 companies were inundated. Of these companies, almost 57% were owned or operated by Japanese companies (Haraguchi and Lall, 2013). In Ayutthaya Province the floods struck when the region was just beginning to recover from the repercussions of the earthquake and tsunami that hit Japan in 2011 (Aon Benfield, 2012). The cost of property damage and opportunity losses of these 804 companies amounts to more than 20 billion US$ and 21 billion US$, respectively (Komori et al., 2012). The manufacturing industry, especially automotive (Fig. 7.17), electronics and electrical appliances were primarily damaged (Chongvilaivan, 2012). Of the impacted carmakers, Toyota was affected more than any other company (Aon Benfield, 2012). Although it required only 42 days to resume operations (in contrast to Honda which needed 174 days to resume its production cycle in the Rojana Industrial Park (Haraguchi and Lall, 2013)), Toyota lost production of 260,000 cars to the floods and suffered a 56% decrease in net revenue or 2.3 billion US$ (Aon Benfield, 2012).

Cascading effects

The prolonged inundation of industrial sites resulted in a loss of production throughout the duration of the floods and longer which disrupted the global supply chain for a number of major industries, and most importantly for automobiles and electronics. Following the flood damage to its plants in Thailand, Carmaker Honda, for instance, had to bring down production by 50% in the United States and Canada until the end of 2011 (Aon Benfield, 2012). According to Chongvilaivan (2012), more than 50% of the manufacturing sector operations could not be restored to usual levels within three months which integrated the loss of property with declines in market shares, competition force and global procurement sustainability.

Fig. 7.17 Inundated automotive factory[4] and submerged cars[5] in Ayutthaya Province

7.3.1.4 Contributing factors

Several factors aggravated the catastrophic consequences of the 2011 Thailand floods. According to Aon Benfield (2012) some of the likely reasons for the floods in addition to excessive rainfall included urbanisation, insufficient drainage and flood protection systems, subsidence, and the possible release of waters from upstream dams.

Consequences on industry were exacerbated by the long inundation times, high-humidity and mold damage to sensitive equipment, raw materials and products, as well as a lack of spare parts. Due to the large spatial extent of the flood, affected industries using similar machinery were in need of the same spare parts at the same time (Aon Benfield, 2012).

7.4   LESSONS LEARNED FROM FLOODS

7.4.1   Systems weaknesses and critical components

In the case of slowly-rising river floods, structural damage to industrial buildings is unlikely. However, non-structural components, such as partition walls, and equipment, stocks and products sensitive to water intrusion or high humidity, are at risk of irreparable damage. Embankments protecting industrial complexes from floodwaters are vulnerable to breaching if they are poorly made.

7.4.2   Potential for propagation

The risk of global cascading effects is high due to weaknesses in the global supply chain (supply and demand) management of industrial facilities. The modelling of

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[4]http://media.cleveland.com/business_impact/photo/an-aerial-view-shows-a-flooded-honda-car-factory-in-ayutthaya-province-thailand-8673128e572576e4.jpg

[5]http://www.brooklyntobangkok.com/wp-content/uploads/2011/11/cars.jpg

industrial facilities as components of a larger system is complex but deserves further attention to avoid cascading economic effects similar to those seen during the Thai floods to occur in future events.

7.4.3   Consequence severity and extent

Losses can be severe due to damage caused by the inundation itself and costs associated with business interruption. Where companies escape a flood unscathed there is still the risk of suffering downtime via supply-chain effects from affected industry that limit the availability of raw materials needed for production. Haraguchi and Lall (2013) note that the automobile sector suffered enormous losses primarily because one company that produces critical components was inundated. As a consequence of the supply and demand disequilibrium in several affected industrial sectors, there was a hike in prices globally.

7.4.4   Protection measures and systems

Improved urban planning that keeps assets out of harm's way, is a tool that should be used to mitigate future losses. In this context, Lebel et al. (2011) propose the use of former agricultural lands near the river basin as new industrial regions. Interestingly, in spite of significant losses many companies indicate that they will not significantly change their investment behaviour. In a survey carried out by the Japan External Trade Organisation 78% of 50 companies that suffered direct flood impact continued to produce in the same location (Haraguchi and Lall, 2013). Only 16% moved their operations to other areas.

Embankments around industrial estates should be built in conformity with worst-case flood severity predictions and best practices for constructing sturdy protection barriers. During the Thai floods embankments failed to protect industrial areas because they were either too low, were made of soil or had discontinuities. The construction of higher embankments is planned in Thailand (Aon Benfield, 2012).

Diversifying procurement sources will help businesses reduce their downtime. Industry has recognised this and following the 2011 floods in Thailand, only a small portion of surveyed industries indicated they would replace their substitute suppliers with their original ones once they had recovered fully from the floods (Haraguchi and Lalla, 2013). Market pressure will force suppliers to seriously consider flood risks in their investment decisions if they do not want to lose customers.

Two dams in Thailand stored two thirds of the total flood volume which mitigated the effect of the flood significantly. Nevertheless, Komori et al. (2012) claimed that the amount of water stored in these dams could have been 20% highter if they hadn't stored the water during June and July. This highlights the importance of seasonal weather forecasting to guide through the trade-off between storing water for dry seasons and releasing it to mitigate flood risk.

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8   Conclusions

Recent events have highlighted the potential for catastrophic natural-hazard impact on critical infrastructures, with consequences ranging from health impacts and environmental degradation to major economic losses due to damage to assets and business interruption. Some of these events have also drawn attention to the risk of cascading effects, either due to accident propagation onto neighbouring infrastructures, or because of interconnectedness between critical infrastructures. The vulnerability introduced into infrastructure systems by interconnectedness is not routinely assessed.

The analysis of case histories across different types of critical infrastructures exposed to different natural hazards highlighted some common lessons learned. For major earthquakes, floods and tsunamis, there is a high risk of multiple and simultaneous impacts at a single infrastructure or to several infrastructures over a potentially large area. These impacts can cause structural damage or affect non-structural components and contents. Where the manufacturing industry has not sustained damage during a natural disaster, it still might have to reduce or stop production due to impacts at suppliers of raw materials or because products cannot be delivered where major transport hubs are affected by the natural hazard.

The study also highlighted the major risk of cascading effects for all analysed critical infrastructures. For Natech accidents involving the release of flammable substances, the risk to facilities nearby is significant due to the high likelihood of ignition. In case of tsunami- or flood-triggered releases, the risk of major consequences is even higher, due to the dispersion of flammable substances over wide areas with the floodwaters. The simultaneous downing of lifelines needed for combating the consequences of such accidents, aggravates the risk of major consequences. The highest priority should therefore be given to preventing these accidents in the first place. For infrastructures that do not house or process hazardous substances, the damage usually translates into potentially severe ripple effects on the economy. Major past events have shown that these effects can reach global proportions, resulting in a shortage of raw materials or intermediate products in the manufacturing industry and causing price hikes.

Emergency response in case of large-scale natural-hazard impact usually suffers from a competition for scarce response resources, where the highest priority is given to preserving human life. In case of hazardous installations with releases, immediate intervention is warranted to protect the population but also to avoid the hampering of emergency response due to hazardous-materials releases. The same applies to recovery activities, where infrastructures essential for serving emergency-response purposes and for mitigating the disaster impact on society will be prioritised. This can lead to prolonged business interruption for infrastructures considered to be lower priority.

The severity of some of the analysed natural hazards was unexpected but many were not entirely unforeseen. Nevertheless, they caused a significant amount of damage and infrastructure service outage. This indicates an underestimation of the risk that resulted in siting in natural-hazard prone areas, insufficient design and a lack of preparedness. In order to avoid future disasters, the vulnerability of key critical infrastructures to natural hazards and the consequences of impact should be determined. Risk analysis is required to understand system weaknesses and to prioritise prevention and mitigation measures. This should be coupled with a cost-benefit analysis. This risk assessment needs to be based on the realistic assessment of a natural hazard's severity that includes, where necessary, climate-change projections.