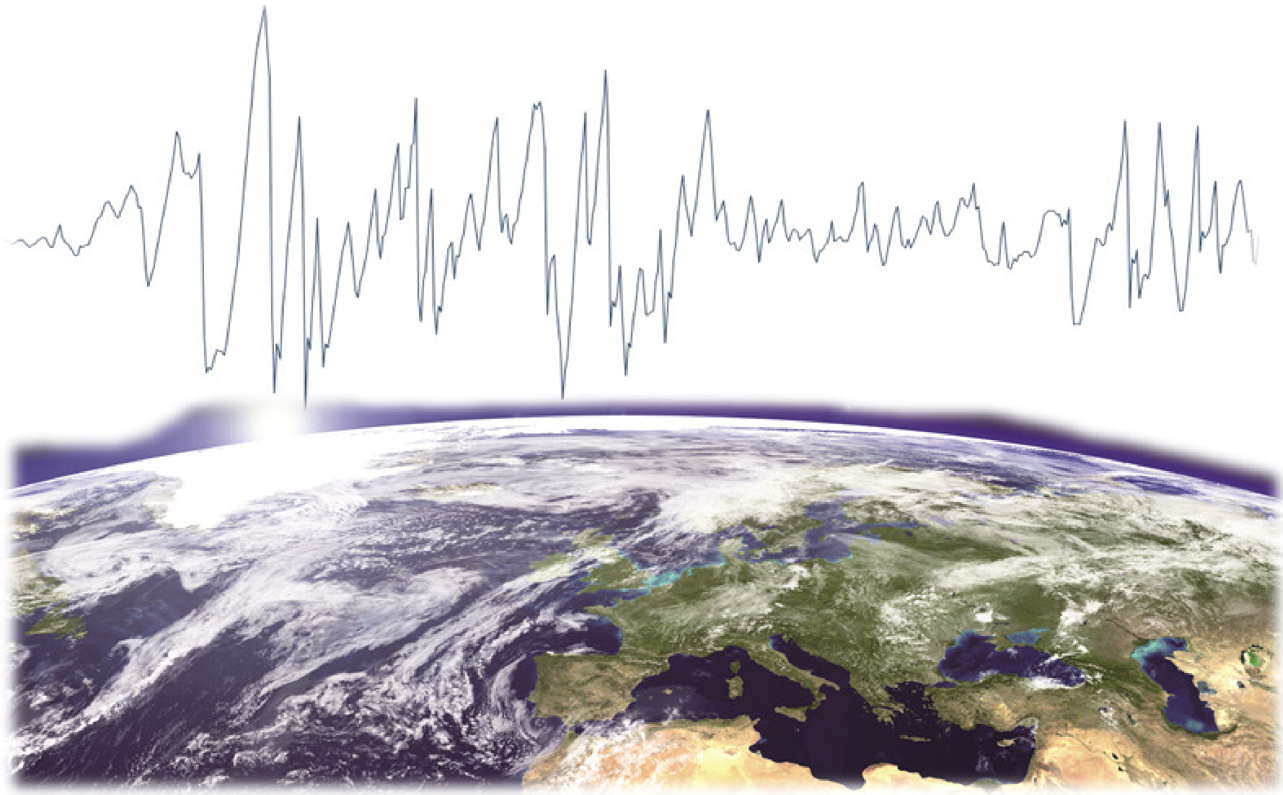

Earthquake Engineering

Capability Statement

Capability Statement: Earthquake Engineering

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Project details throughout this document may have been utilised the services of companies previously acquired by Royal HaskoningDHV.

Royal HaskoningDHV, headquartered in Amersfoort (the Netherlands), is one of Europe's leading project management, engineering and consultancy service providers. The company has offices in 35 countries in Europe, America, Australia, Asia and Africa. Over 7,000 professionals deliver first class solutions in the fields of aviation, maritime, waterways, infrastructure, planning, strategy, water technology, rivers, deltas, coasts, buildings, industry, energy and mining.

A group of 30 specialists, geotechnical and structural engineers, among them is actively involved in designing buildings, dams, dikes, bridges and other structures for earthquake loading all over the world. A variety of services in different stages of projects can be offered. Seimic expertise ranges from quickscans to detailed in-depth studies. Royal HaskoningDHV always keep an eye on the total project requirements.

The quality management system of HaskoningDHV Nederland B.V., part of Royal HaskoningDHV, has been approved against ISO 9001.

1 Quickscan

A company, municipality or (governmental) institution is generally responsible for a group of structures that may be affected by earthquake loading. If a seismic hazard assessment has been carried out for a specific area rather than a single structure, Royal HaskoningDHV can perform a quickscan in order to identify and mitigate the most prominent risks. For that purpose the flow chart in Figure 2 can be followed.

2 Response spectrum analysis

An elastic response spectrum characterizes the maximum response of structures with a certain amount of internal damping to earthquake loading at a specific location with defined subsoil conditions. An example of such a spectrum is plotted in Figure 1. Different types of earthquakes can be distinguished, such as tectonic and induced by human activity, all with different characteristics. The elastic response spectrum, which is derived from the applicable building code (e.g. EN 1998-1 in Europe) or extracted from a dedicated seismic hazard assessment, is the basis for the lateral force method or a response spectrum analysis. While the former method quickly produces results for simple structures or structures that can be simplified to single-degree-of-freedom (SDOF) structures, the response spectrum analysis is basically a linear-elastic analysis carried out in the frequency domain. It involves all natural frequencies of the structure and projects them on the elastic response spectrum. The resulting structural behavior due to the specified earthquake loading is generally obtained by combining the modal contributions by the SRSS or CQC method. Even impulsive and convective (sloshing) modes of liquids inside containers can be accounted for. Royal HaskoningDHV uses the FEM software DIANA to perform those tasks. For piping the software AutoPipe is deployed.

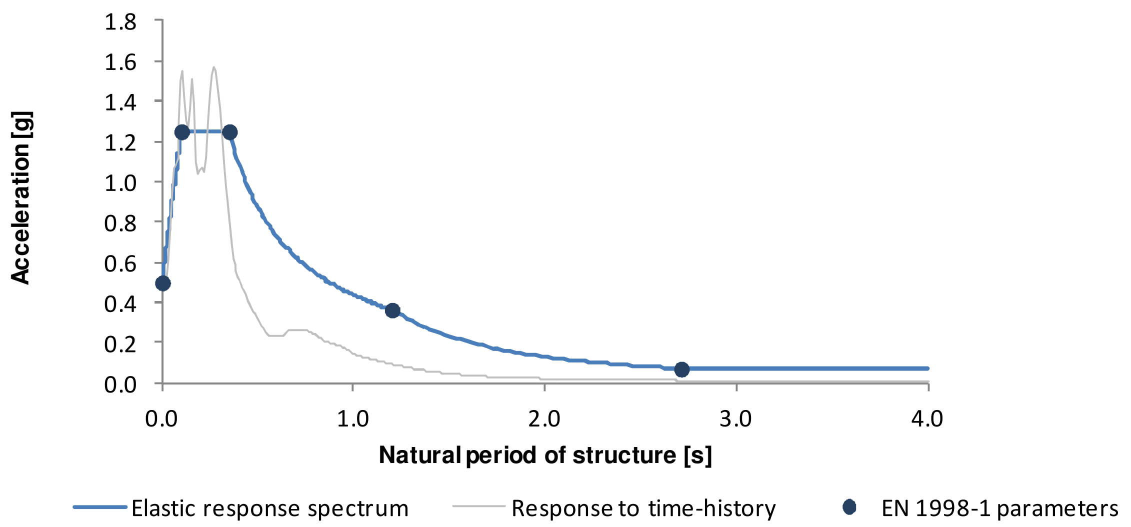

Figure 1: example of 5% damped horizontal elastic response spectrum based on the formulation in EN 1998-1 and example of the 5% damped elastic response to a single time-history

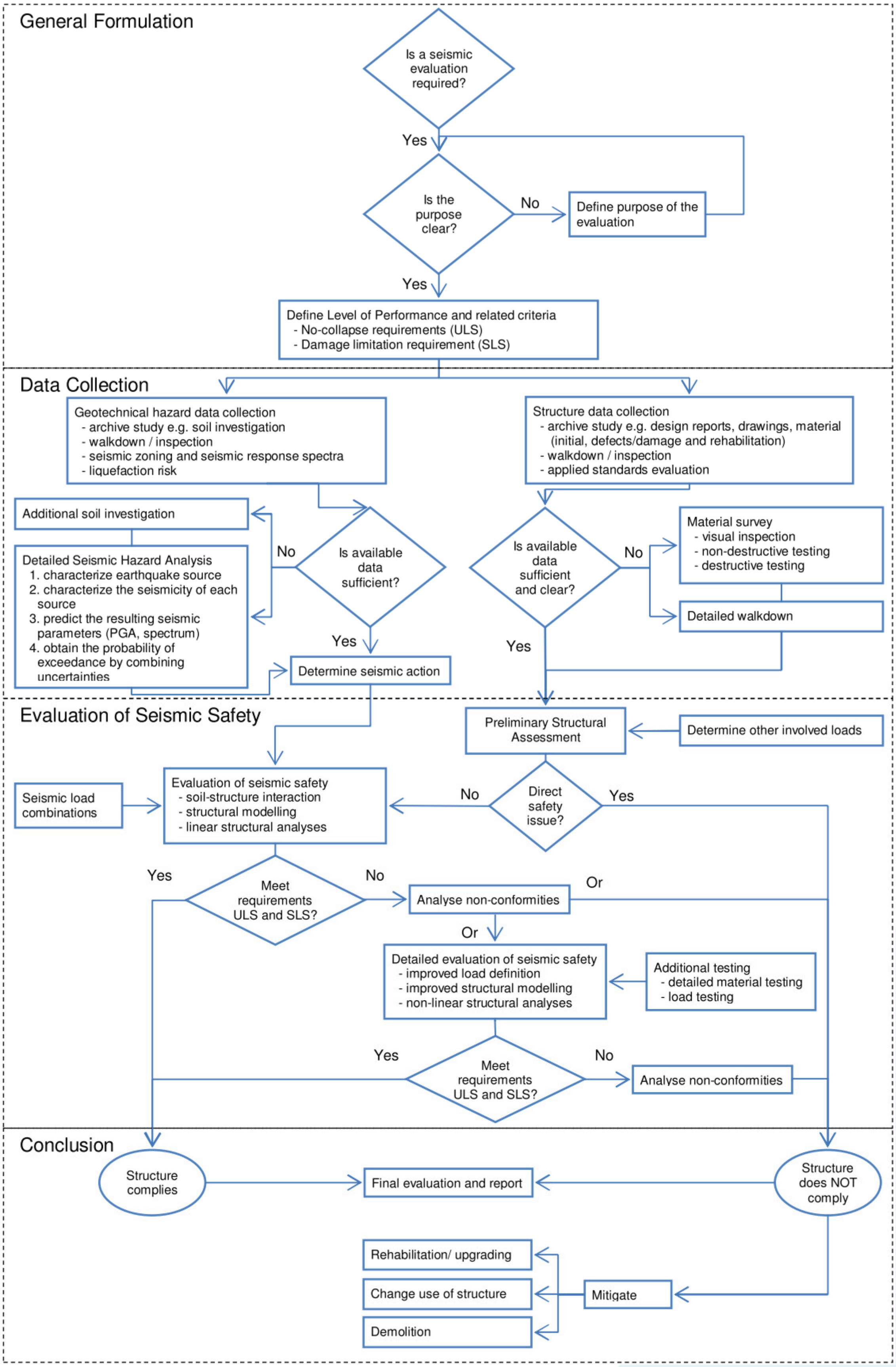

Figure 2: flow chart for a quickscan of risks related to earthquake loading

3 Time-history analysis

Historical earthquakes, both tectonic and induced, have been recorded in so-called time-histories, examples of which are depicted in Figure 3. Time-histories can be transformed to a response spectrum by determining the maximum response to the time-history of a series of SDOF systems with defined natural frequencies and damping ratios. Conversely, artificial time-histories can also be generated to match an elastic response spectrum. In a time-history analysis the structure is loaded at the base by the accelerations of the time-history and its structural response is evaluated in the time domain. Non-linear elasticity, hysteretic and dissipative behaviour can be taken into account as opposed to a response spectrum analysis. Again the FEM software DIANA is deployed to perform those tasks.

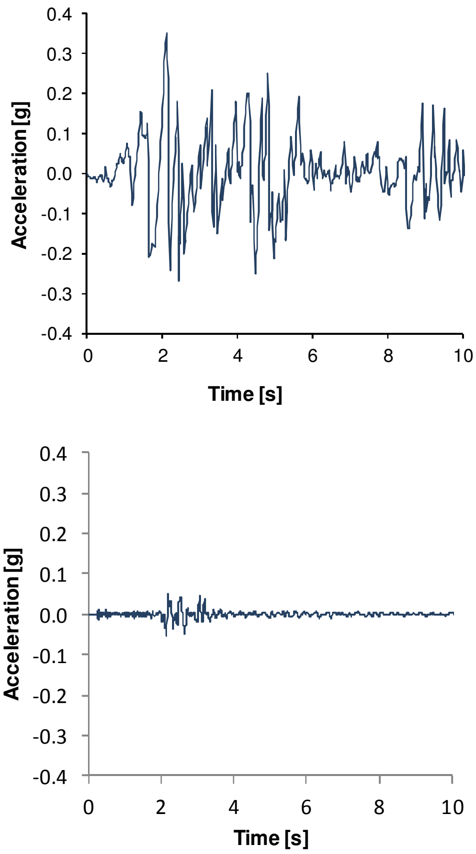

Figure 3 : example of recorded tectonic earthquake time-history (El Centro, Mexico 1940, top figure) and induced earthquake time-history (Westeremden, the Netherlands 2012)

4 Push-over analysis

In a push-over analysis the ductile behaviour of the structure is modeled by applying a gradually increasing load pattern corresponding to the primary mode shape up to failure of the structure or exceedance of limiting plastic strains. Non-linear behavior is accounted for by introducing plastic hinges which is monitored during the analysis. This provides a basis for investigating the post-elastic behavior and dissipative properties at a local level, which is more precise than using global behaviour factors from a building code. This insight is particularly useful for irregular structures where ductility demand can be concentrated in certain parts of the structure. The push-over analysis is used to determine the displacement capacity of the structure, i.e. the displacement of the deck or roof at which limiting plastic strains will be exceeded. The actual displacements during the earthquake are subsequently determined using a demand analysis, e.g. capacity spectrum or substitute spectrum method using proprietary software. Royal HaskoningDHV can use advanced modeling techniques and material models in SAP and DIANA to carry out a push-over analysis. A key-input to the pushover analysis is moment-curvature analysis of the plastic hinges, an area in which Royal HaskoningDHV also have significant experience.

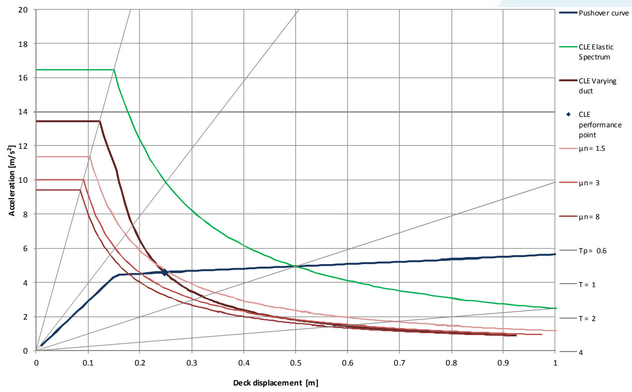

Figure 4 example of push-over analysis results

5 Soil-structure-interaction

Royal HaskoningDHV can assess the subsoil conditions based on available soil investigation results or propose the extent and type of soil investigations to be carried out. This not only serves the correct definition of the joint behaviour of subsoil and large structures under the applicable earthquake conditions, so-called soil-structure interaction, but also identifies potential liquefaction risks. Liquefiable soils may destabilize structures, both on raft and pile foundations. Royal HaskoningDHV has models at hand to account for soil-structure interaction.

Experts from Royal HaskoningDHV have also been working for solutions for earthquake triggering landslides and ground or slope movements. Underground structures such as tunnels and caverns under seismic excitations, are also a part of our studies. PLAXIS and PHASE software can be used for the analyses.

Furthermore, the Turkish office is available for detailed site investigations (seismic exploration works, geophysical studies, advanced soil sampling, etc.) with its experts and equipment.

6 Liquefaction and cyclic softening

Liquefaction is the phenomenon which is observed when there is loss of strength in saturated and cohesionless soils because of increased pore water pressure and hence reduced effective stresses due to dynamic loading. As a result of liquefaction, the soil merely behaves like a fluid mass with hardly any shear strength, which can lead to serious damage of structures constructed in such soils. Liquefaction induced ground failures include loss of bearing strength, lateral spreading, and flow failures, which may cause many engineering problems such as foundation failures, damage to utilities, slope failures, landslides and large displacements of earth dams.

Lateral spreading can occur with very small slope angles of 0.3 to 5 % and will induce large lateral loads on any piles penetrating the liquefiable layer. If a non-liquefying layer is present on top of a liquefying (and laterally spreading) layer, this material will exert an even larger load on any installed piles. As the lateral spreading will be in the order of meters, sufficient soil strains are mobilised to generate full passive earth pressures.

Cohesionless soils that have a factor of safety for liquefaction larger than 1.0 may still develop excess pore pressures during an earthquake. The amount of pore pressure build-up is linked to the factor of safety for liquefaction. Relevant relations are based on the original work by H.B. Seed.

As for cohesionless soils, saturated plastic silts and clays have the potential for strength loss and rapidly increasing strains during dynamic loading as well. This is referred to as cyclic softening. Royal HaskoningDHV has ample expertise available to be able to evaluate the potential for liquefaction and/or cyclic softening and design remedial measures when needed.

7 Displacement based design

With respect to slope stability, the most commonly used pseudo-static analyses ignore the cyclic nature of the earthquake and treat it as a constantly applied additional static load on the slope. Clearly, the difficulty with the pseudo-static approach is that the seismic acceleration varies with time and, as a result, the factor of safety in reality will vary dramatically both above and below the static factor of safety. It is widely accepted that the calculated factor of safety can fall below 1.0, but this does not necessarily mean that the slope will catastrophically collapse. Indeed, if the pseudo-static factor of safety drops below 1.0 for a mere fraction of a second, then the slope deformation will be limited.

For this reason, in most seismic slope designs a performance based design approach is adopted for cases where the pseudo-static safety factor cannot easily be met (or would result in an uneconomic design solution). In the displacement or performance based design approach the seismic induced slope displacements are assessed and it is checked whether the resulting displacements can still meet the project's performance requirements. It is checked whether the seismic induced displacements can still be safely accommodated by the slope or the structures that might exist within the zone of influence. The performance based approach results in a fit-for-purpose and economic design solution.

Seismic induced slope deformations can be assessed using real or 'developed' time histories (accelerograms) by means of a full dynamic analysis (using e.g. PLAXIS FEM software).

Alternatively it could be considered to use the Newmark (or similar) method to estimate the slope deformations in cases that the pseudo-static slope safety factor is less than unity. In its simplest form, the Newmark approach models the moving part of the slope as a rigid-plastic block which starts to move (accelerate) along its base when a critical (yield) acceleration is exceeded. The ratio between the design peak acceleration and the yield acceleration is a measure for the corresponding block displacement. Figure 5 illustrates the principle of the displacement calculation, based on a double integration of the 'excess' acceleration (the part of acceleration that exceeds the yield acceleration and makes the block moving).

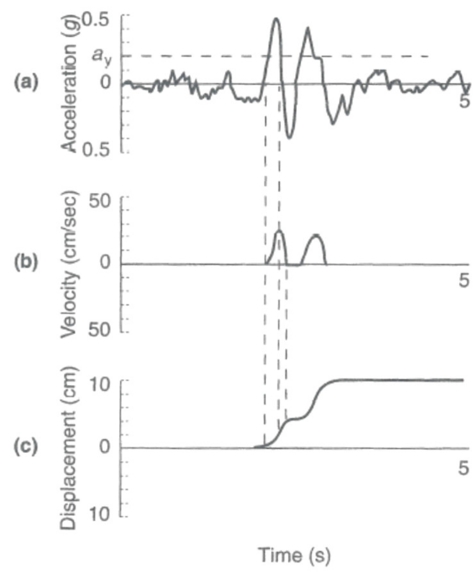

Figure 5 : Principle Newmark analysis

In reality, different parts of a slope will have different yield accelerations. The yield acceleration normally increases with increased depth of the slip surface, unless there is a weak (e.g. liquefying) layer present in the slope. Obviously, the lower the yield acceleration for a particular slope section, the larger the earthquake induced displacement will be.

Royal HaskoningDHV has access to dedicated software programs to perform performance based seismic slope designs, like PLAXIS and SHAKE2000. Alternatively, simplified Newmark analyses may be performed based on e.g. Ambraseys and Menu.

It is finally noted that seismic displacement is not only resulting from the inertia force acting on the slope. Liquefaction induced displacement (cyclic shake down) settlement can also significantly contribute to the total seismic induced displacement. Any performance based design approach should also account for cyclic softening and seismic induced pore pressure build-up.

8 Detailing of structures

Proper detailing of structures is of paramount importance to withstand earthquake loading. In general the first step is to decide on the level of ductility to be created in the structure. EN 1998-1 distinguishes between low, medium and high ductility, with associated rules for detailing and suitable materials. An example of additional detailing for earthquake loading is shown in Figure 6. Design for low ductility is possible in low seismicity areas.

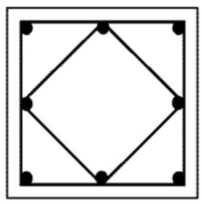

Figure 6 : example of additional concrete column rebar for earthquake loading according to EN 1998-1

For higher seismicity specific parts of the structure must be appointed to safely absorb the imposed earthquake energy. In some cases commercially available seismic base isolation systems may be applied instead, which basically enlarge the first natural period of the entire structure to such a level that the overall seismic response is reduced drastically.

9 Quantitative risk analysis

Quantitative risk analysis (QRA) is an important method for calculating the potential risks due to the handling, storage, et cetera of dangerous goods. The potential risks are a product of probability of failure and possible effects. The occurrence of earthquakes may influence the probability of failure and therefore the potential risk of industries using dangerous goods. Insight in the potential risks and the influence of the possibility of earthquakes is necessary for preventing major hazards and taking the right measure to prevent major hazards.

Royal HaskoningDHV have large experience in quantitative risk assessments related to all kinds of (major) hazards for industry and transport of dangerous goods. The additional (external) risk of an installation by adding the probability of failure due to an earthquake (or flood) to the failure frequency in the existing QRA of the facility can also be calculated. This additional failure frequency will be based on the studies listed in the paragraphs above.

Royal HaskoningDHV have extensive experience with QRA for the process and chemical industry as well as the oil and gas industry (upstream and downstream). Reference is made to the list of projects below.

Royal HaskoningDHV also have extensive experience with performing second opinion checks on QRA's prepared by others and with performing effect calculations for potential incidents.

Client	Project
Sonneborn	Chemical plant with (a.o.) SO3 (Amsterdam)
Croda	Chemical plant
Caldic	Chemical plant
Stahl Europe	Chemical plant
BASF	Chemical plant
Sachem Europe	Fine chemicals plant
Dr. W. Kolb Nederland B.V.	Chemical plant
Tanatex B.V.	Chemical plant
Century Aluminium Vlissingen B.V.	Anode production
Various (a.o. Flora Holland, 3 refrigeration plants in Waalwijk)	Ammonia cooling plants
Various (a.o. Bosman; C1000; ATM; Norit; KLM Cargo)	Storage of dangerous goods (PGS-15)
Various (a.o. Utrecht, Roermond)	LPG filling stations
Various (a.o. Contrall, Tango)	Petrol filling stations
Various (a.o. Argos)	Fuel depots
Various (a.o. Vopak, Rubys)	Chemicals depots
Various (a.o. Verbrugge, Waalwijk)	Container terminals
Port of Rotterdam	Various QRA's related to shipping
BP Refinery Rotterdam	Refinery QRA
Various (a.o. NAM, Vermilion, TAQA)	QRA's for on and offshore oil and gas installations
DSM	Idem
Teijin	Chlorine
Road	Supercritical CO2 transport
Various pipelines (e.g. Supercritical CO2, Natural Gas, ethylene oxide	QRA's
Accsys	Wood treatment (Acetic acid anhydride, Acetic acid)
KLM E&M	Galvano, Jet fuel, PGS15
Various (a.o. 4Gas B.V., Petro Vietnam and Aqaba)	LNG terminal

Table 1: recent project references of Royal HaskoningDHV with respect to QRA's

Royal HaskoningDHV

10 Projects

Royal HaskoningDHV has applied earthquake engineering in many projects around the world over the past decades. The recent projects listed in the table below demonstrate the company's current capability with respect to earthquake engineering.

Project	Client	Earthquake characteristics	Lateral force method	Response-spectrum analysis	Linear-elastic time-history analysis	Non-linear elastic (hysteretic/dissipative) time-history analysis	Artifical time-history generation	Push-over analysis	Soil-structure-interaction and liquefaction analysis	Displacement based design	Detailing according to EN 1998
BUILDINGS											
Ashgabat International Airport, control tower and terminal, preliminary design, 2013	Polimeks	Tectonic PGA = 0.60*g	•	•		•			•		
Ashgabat International Airport, utility buildings and infrastructure, detailed design, 2013	Polimeks	Tectonic PGA = 0.60*g	•	•							
Bedas Building Structural Strengthening Istanbul, 2007	BEDAS	Tectonic PGA = 0.40*g		•	•	•			•		
Bursa Modern Housing Project, 2010	Sinpas REI	Tectonic PGA = 0.40*g				•			•		
Control tower Flaming Airport, Bonaire 2013	Rijkswaterstaat Netherlands	Tectonic PGA = 0.15*g	•								
Dutch Embassy building Kabul, 2010	Dutch Ministry of Foreign Affairs	Tectonic PGA = 0.40*g		•							
Dutch Embassy Islamabad,Pakistan, 2003	Dutch Ministry of Foreign Affairs	Tectonic PGA = 0.39*g		•			•	•			

Royal HaskoningDHV

Project	Client	Earthquake characteristics	Lateral force method	Response-spectrum analysis	Linear-elastic time-history analysis	Non-linear elastic (hysteretic/dissipative) time-history analysis	Artifical time-history generation	Push-over analysis	Soil-structure-interaction and liquefaction analysis	Displacement based design	Detailing according to EN 1998
Dutch Embassy New Delhi, India, 2005	Dutch Ministry of Foreign Affairs	Tectonic PGA = 0.24*g						•			
Dutch Embassy Teheran, Iran, 2006	Dutch Ministry of Foreign Affairs	Tectonic PGA = 0.37*g	•								
Eroglu Kagithane Housing Project,2013	Eroglu	Tectonic PGA = 0.40*g							•		
Firestation Bonaire 2013	Rijksgebouwen-dienst Netherlands	Tectonic PGA = 0.15*g	•								•
Istanbul 1453 Housing Project, 2013	Agaoglu	Tectonic PGA = 0.40*g				•			•	•	
Marmarapark Shopping Mall, 2012	ECE Investments	Tectonic PGA = 0.40*g				•			•	•	•
Tarin Kowt Airfield, 2010	Deutsche Gesellschaft für Internationale Zusammenarbeit	Tectonic PGA = 0.24*g	•								•
Tsunami Escape, 2008	Dutch Embassy	Tectonic PGA = 0.30*g	•								
Vinamilk smart storage	Vinamilk	Tectonic PGA = 0.09*g		•							
INDUSTRY, ENERGY & MINING											
Angola 159,000 m3 LNG, detailed design, 2009	Toyo Kanetsu K.K.	Tectonic PGA = 0.03*g (OBE), PGA = 0.10*g (SSE)		•	•		•		•		
Bartin Cement Plant Turkey, 2009	Bilim Makina INC	Tectonic PGA = 0.40*g							•		
BTC Crude Oil Pipeline, 2013	Tekfen Construction	Tectonic PGA = 0.40*g							•		

Royal HaskoningDHV

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Dutch Lady Ha Nam	Royal Friesland Campina	Tectonic PGA = 0.12*g	•								
Greenfield Brewery Kilinto – Addis Ababa	Heineken Breweries Share Company	Tectonic PGA = 0.82*g	•								
Habeco	Habeco	Tectonic PGA = 0.08*g	•								
Jordan Dike 18 Seismic Stability, 2008	Àrab Potash Company	Tectonic PGA = 0.30*g				•			•	•	
L'Oreal Jababeka	PT. Yasulor Indonesia	Tectonic PGA = 0.30*g	•								
Lahad Datu LNG, bid design, 2012	Sato Kogyo Malaysia	Tectonic PGA = 0.12*g (OBE), PGA = 0.22*g (SSE)	•								
Monas Project, detail design of production building, boiler building, pipe bridge & tank farm	PT. Rckitt Benckiser Indonesia	Tectonic PGA = 0.35*g	•								
Morocco Samir Mohammadia Refinery Plant, 2007	Tekfen Consulting	Tectonic PGA = 0.40*g							•		
MSD Indonesia Packaging Project	PT. Schering Plough Indonesia	Tectonic PGA = 0.10*g	•								
Nestle Kejayan BO Plan Expansion	PT. Nestle Indonesia	Tectonic PGA = 0.15*g	•								
Nestle Panjang, Filling Room and Dry Mix Extension	PT. Nestle Indonesia	Tectonic PGA = 0.20*g	•								

Royal HaskoningDHV

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NW380 planned electrical powerline, 2013	TenneT Netherlands	Induced PGA = 0.43*g		•	•						
OTSUKA New SVP Factory Indonesia	PT. Otsuka Indonesia	Tectonic PGA = 0.15*g	•								
P9 pumping station, pipe works and canal, 2014	Dead Sea Works	Tectonic PGA = 0.29*g	•						•	•	
Pepsico	Suntory Pepsi Viet Nam	Tectonic PGA = 0.11*g	•								
Poti Tank Storage Space, Georgia, 2002	Üstay Construction INC	Tectonic PGA = 0.40*g							•	•	
Quang Ngai Brewery	Sabeco	Tectonic PGA = 0.08*g	•								
Quickscan Chemical plant (pipeline) Delftzijl, 2013	AkzoNobel Netherlands	Induced PGA = 0.43*g		•							
Quickscan salt pipeline Delfzijl-Midwolda, 2013	AkzoNobel Netherlands	Induced PGA = 0.43*g		•							
Radar Merauke in Timika, Merauke, Saumlakki	PT. Ebdesk Indonesia	Tectonic PGA = 0.30*g	•								
Rijeka Container Terminal, 2013-2014	Port of Rijeka Authority	Tectonic PGA = 0.40*g							•		•
Song Lam Brewery	Sabeco	Tectonic PGA = 0.11*g	•								
Vinamilk Dielac II	Vinamilk	Tectonic PGA = 0.08*g		•							
Wrigley	Wrigley	Tectonic PGA = 0.09*g	•								

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Azerbaijan Shah Deniz Project, 2003	Tekfen Construction	Tectonic PGA = 0.40*g							•		
Fujairah VLCC piping, 2014	Port of Fujairah	Tectonic PGA = 0.12*g	•								
Bukavu piping project, 2013	Heineken	Tectonic PGA = 0.11*g	•								
Kisangani piping project, 2013	Heineken	Tectonic PGA = 0.11*g	•								
Kigali piping project, 2012	Heineken	Tectonic PGA = 0.11*g	•								
Adana Power Plant, piping, Turkey, 2001	Siemens	Tectonic PGA = 0.50*g	•								
INFRASTRUCTURE											
Ankara Pozanti Highway Engineering Structures, 2009	Tekfen Construction	Tectonic PGA = 0.40*g				•			•		
Baku By-pass Highway Slope Stability Rehabilitation, 2008	Makyol-Copli JV	Tectonic PGA = 0.40*g				•			•		
Bursaray B line LRTS, Turkey, 2006	Bursa Metropolitian Municipality-Siemens-Tekfen-Tuvasas JV	Tectonic PGA = 0.40*g				•			•		
Dubai Iconic Bridge, 2008	Nakheel	Tectonic PGA = 0.15*g	•						•		
Istanbul Metro System, 90 km, 6 Lines, Turkey, 2009	Istanbul Metropolitian Municipality	Tectonic PGA = 0.40*g							•		

Royal HaskoningDHV

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Sabiha Gokcen International Airport Second Runway, 2011	ARUP	Tectonic PGA = 0.40*g				•			•		
The Bosphorus Rail Tube Crossing Istanbul Turkey, 2013	Avrasya Consulting	Tectonic PGA = 0.40*g							•		
Avcilar-Ambarli Landslide Remediation Project, 2005	Istanbul Metropolitian Municipality	Tectonic PGA = 0.40*g				•			•		
Strait Crossing European Under Pass, 2011	Yapi Merkezi-SKEC JV	Tectonic PGA = 0.40*g				•			•		
MARITIME											
Ambon Bay Land Reclamation	PT. Karya Unggulan Gemilang	Tectonic PGA = 0.53*g							•		
Baku New International Sea Trade Port	Ministry of Transport	Tectonic PGA = 0.22*g	•						•	•	
Buyukcekmece Marina, Istanbul Turkey,2010	Marina Istanbul	Tectonic PGA = 0.40*g				•			•		
Callao Muelle Norte Port structures, Peru 2012	APMT	Tectonic PGA = 0.43*g		•				•	•	•	•
Candarli Port, Izmir Turkey, 2009	DLH-Altinok Consulting	Tectonic PGA = 0.60*g				•			•	•	
Damen Song Cam Shipyard	Damen Song Cam	Tectonic PGA = 0.13*g	•								
Dubai Palm Deira Marine Waterfront Structures, 2009	Nakheel	Tectonic PGA = 0.15*g			•				•	•	

Royal HaskoningDHV

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GOA Shipyard	GOA Shipyard Ltd	Tectonic PGA = 0.16*g		•							
Kargı Earth Dam Project, 2010, Turkey	Makyol	Tectonic PGA = 0.40*g							•		
Lamongan shipyard 2010 – 2011		Tectonic, PGA = 0.30*g		•							
Land Reclamation Tanjung Bunga Makassar, Indonesia 2013	PT. Gowa Makassar Tourism Development	Tectonic, PGA = 0.20*g							•		
Mersin Port, Turkey 2009	DLH-Altinok Consulting	Tectonic PGA = 0.45 g				•			•		
Pluit City Land Development Project, Indonesia 2013	PT Muara Wisesa Samudra	Tectonic, PGA = 0.30*g							•		
Ruwals Takreer oil loading jetties, detailed design 2010-2011	GS E&C Corporation	Tectonic PGA = 0.11*g		•							
Toros Tarim Ceyhan Mobile Erection Area, 2010	Tekfen Construction	Tectonic PGA = 0.40*g				•			•		

Table 2: recent project references of Royal HaskoningDHV with respect to earthquake engineering

Notes:
g = gravity acceleration (9.8 m/s^2)
PGA = peak ground acceleration at surface level
ULS = ultimate limit state (near collapse)
SLS = serviceability limit state (serious damage)
OBE = operation basis earthquake
SSE = safe shutdown earthquake