ABSTRACT

Natural gas production from the Groningen field in the Netherlands is causing induced earthquakes. The safety of the local population and the likelihood of damage to the buildings, which have not been designed and constructed with a consideration of seismic loading, is being quantified through a probabilistic seismic risk model. This model is calibrated to local conditions and captures all major sources of aleatory variability and epistemic uncertainty.

For a given risk assessment period and gas production scenario, the spatial-temporal development of reservoir pore pressure changes within the field is forecast and history-matched against historical gas production data. Given these pressure changes, reservoir compaction maps are developed as input to the seismological model. Monte Carlo sampling is then used with the seismological model to develop a simulated earthquake catalogue of induced earthquakes, epicentral locations, origin times and magnitudes, and contributions from aftershocks. Estimates of ground motion parameters at the location of the buildings in the exposure model are then sampled for each event in the earthquake catalogue, using the ground-motion prediction equation (GMPE) that has been developed for the field. The probability of a given damage or collapse state (either partial or complete), conditional on the aforementioned ground motion parameters, is then estimated for a given building type at a given location using the fragility model. This paper describes the development of the aforementioned fragility model, as well as the consequence model required to calculate the fatality risk.

Keywords: Induced seismicity; seismic risk; collapse modelling; fragility model; fatality model.

1. INTRODUCTION

Gas production in the Groningen field in the northern Netherlands is inducing earthquakes, the largest of which to date was the magnitude ML 3.6 (M 3.4) Huizinge event of August 2012. In response to this induced seismicity, Nederlandse Aardolie Maatschappij B.V. (NAM) has been developing a comprehensive seismic hazard and risk model for the region, which comprises the entire gas field plus a 5 km buffer zone onshore.

1.1 Components of the Risk Model

The main elements of the risk model are the seismological model, the ground-motion prediction equation (GMPE), the exposure model, the fragility functions and the consequence model (Figure 1). The seismological model developed for the Groningen field (Bourne and Oates 2017; Bourne et al. 2017) provides the joint conditional probability distributions of earthquake origin times, epicenters, and magnitudes given the reservoir deformations induced by pore-pressure decreases (due to gas production). A ground-motion model has been specifically calibrated to the Groningen conditions as described in Bommer et al. (2017), and combines predictions of horizontal 5%-damped spectral
accelerations (at periods from 0.01 to 5 seconds) at a buried rock horizon with non-linear frequency-dependent amplification factors assigned to 160 zones across the field. The exposure model is a probabilistic model that provides, for each unique building in the region, the probability of a range of structural systems. A significant effort has also been made to remove the uncertainty in the exposure model through building inspections. Fragility functions for the estimation of the probability of damage and collapse have been developed for all structural systems in the exposure model (Crowley et al. 2017), using numerical models that have been calibrated with the results of an extensive experimental testing campaign. Finally, the consequences that are currently being modelled in the risk engine include damage and casualties; for the latter, a model that relates the probability of death given collapse has been developed for all of the structural systems.

![Figure 1. Components of the risk model (figure courtesy of Stephen Bourne)](image)

### 1.2 Implementation of the Risk Model

The risk model is evaluated by Monte Carlo sampling of the aleatory variability within the causal sequence of conditional probability models (Figure 2) and builds on the engine developed for the earlier hazard models (Bourne et al. 2015). For a given risk assessment period and a given gas production scenario, the spatial-temporal development of reservoir pore pressure changes in the field is forecast. Given these pressure changes, the seismological model is used to forecast probability distributions for the total number of induced earthquakes (together with their epicentral locations, origin times and magnitudes, including contributions from aftershocks). By sampling these earthquake distributions, a single simulated earthquake catalogue can be created, and for every simulated earthquake exceeding a given magnitude threshold, the ground motion model is used to forecast the probability distribution of base rock motions and near-surface amplifications across a dense surface grid of observation points. Sampling these ground motion distributions creates a single catalogue of simulated ground motions. For each ground motion event in this simulated catalogue of ground motions, the fragility model is used to forecast the probability distribution of building damage states (considering the structural systems located at each observation point). Sampling these probability distributions for each exposed building yields a catalog of damaged buildings. Given this catalog of damaged buildings, the consequence model forecasts the probability distribution of consequences for each exposed building or individual. Sampling these consequence distributions creates a single catalogue of simulated consequences. Repeating this sampling process a sufficiently large number of times yields a collection of simulated catalogues that closely approximates the consequence probability distribution. Risk metrics that summarize this distribution are then readily computable.

### 1.3 Risk Metrics

This paper presents a methodology for developing fragility and consequence models for the estimation of three risk metrics: “local personal risk” (LPR), group damage, and group risk. As discussed in Jonkman et al. (2003), there are a number of established fatality risk metrics, including Individual Risk (the probability that an average unprotected person, permanently present at a certain location, is killed due to an accident resulting from a hazardous activity). This metric is often referred to as Location Risk and has been adopted by the Dutch Ministry of Housing, Spatial Planning and Environment when setting risk standards for the Netherlands. In early 2015, an advisory committee (Commissie Meijdam)
was established to advise on risk policy related to Groningen earthquakes, including the selection of risk metrics. The first advice of this committee was that an inside local personal risk (ILPR) metric, defined as the annual probability of fatality for a hypothetical person who is continuously present without protection inside a building, should be evaluated for all buildings in the Groningen gas field. ILPR differs from the aforementioned Location Risk metric in that it refers to the area inside a building rather than a single arbitrary location inside or outside buildings. Given that unreinforced masonry buildings also pose a significant threat to people that are outside buildings (due to out-of-plane failure of lightly loaded walls, chimneys and parapets), the outside local personal risk is also calculated, and is weighted by the probability that the aforementioned continuously present hypothetical person would be outside at the time of the earthquake, in order to obtain a single local personal risk (LPR) value per building.

In 2016 the Dutch Ministry of Economic Affairs (MEA) also requested the forecast of group risk for damage (so-called Maatschappelijk Risico (Schade)). To meet this request, F-N curves that present the annual frequency of exceedance against number of damaged buildings have been calculated using the fragility functions for damage states DS2 and DS3 presented herein. Group risk for fatalities can also be calculated using the input models presented herein, by combining the inside and outside LPR by the average number of people present in and around the buildings during the day and night, as provided in the exposure model.

![Figure 2. Schematic representation of the Monte Carlo risk calculation process](image)

**2. OUTLINE OF METHODOLOGY FOR FRAGILITY AND CONSEQUENCE MODELS**

The methodology presented herein attempts to use a predominantly analytical approach, that is augmented where possible with empirical and experimental data, to estimate both fragility and consequence models for estimating damage, local personal risk and group risk (Figure 3). The main causal pathways for loss of life that are currently being considered include the following: being hit by the collapse of a chimney outside of the building, or being hit by the debris caused by different structural collapse states of the building (both inside and outside) brought about by the global dynamic response of the structure to an input acceleration.

In order to model the dynamic response of the population of buildings in the Groningen region, the buildings have been classified into classes of types that have similar structural and architectural
characteristics. Once these typologies have been identified, at least one real representative building from the region for each is found (so-called index building) and the structural drawings are used to develop a multi-degree-of-freedom (MDOF) numerical model of the structural system together with the predominant non-structural elements (such as partition walls and external façade walls). However, the computational effort associated with running nonlinear dynamic analyses of many such numerical models (around 50 different types of building have been defined for the region of Groningen), each subjected to tens of records, was judged to be too high to allow fragility functions to be directly developed from these analyses. Therefore, a simplified single-degree-of-freedom (SDOF) equivalent system approach has been used instead to analytically represent each typology.

For the global response, nonlinear dynamic analysis of the MDOF numerical models using records with increasing intensity has been employed to produce the SDOF backbone capacity curves and to identify the consequences of different collapse mechanisms. A large suite of records was then utilised in the nonlinear dynamic analyses of these SDOF systems to model the record-to-record variability, and regression analysis is used to relate various ground shaking parameters to the nonlinear response in order to produce the fragility functions. Consequence models based on the extent of partial and complete collapse debris observed in the MDOF numerical analyses are then developed. A study of the collapse of chimneys of unreinforced masonry (URM) buildings from a number of earthquakes has been undertaken by Taig and Pickup (2016), leading to empirical fragility functions and consequence models for chimneys.

![Diagram](image)

Figure 3. Outline of the methodology to develop fragility and consequence models, as presented herein

3. DAMAGE AND COLLAPSE FRAGILITY FUNCTIONS

3.1 Seismic Response of Buildings

Nonlinear dynamic analysis (using a set of 11 triaxial ground motions) of the majority of the index
Buildings have been undertaken using LS-DYNA (LSTC 2013), ELS (ASI 2017) and SeismoStruct (Seismosoft 2017), and the results are fully presented in Arup (2017) and Mosayk (2017d). For some of the stronger buildings (constructed in reinforced concrete, steel or timber), nonlinear static analysis has been performed, as described in Mosayk (2015a). It is important to note that the aforementioned software tools have been validated and/or calibrated for the seismic analysis of Groningen buildings using the results of a large number of experimental tests (Graziotti et al. 2015; 2016a; 2016b; 2016c; 2017a; 2017b; 2017c; Tomassetti et al. 2017; Correia et al. 2017) as documented in Mosayk (2014; 2016; 2017a; 2017b; 2017c; 2017e) and Arup et al. (2015; 2016a; 2016b; 2017).

Plots were developed using either the nonlinear static pushover curves or the hysteresis loops of all nonlinear dynamic analyses, together with points representing the peak base shear and corresponding attic floor (i.e. highest level in the building before the roof) displacement in each direction of the building, for each analysis (Figure 4). Shear and displacement response time-histories of MDOF structural systems are not necessarily fully in-phase, particularly when multiple modes of vibration or failure mechanisms are activated during the response of a given structure (a phenomenon that is further accentuated when the structure is pushed into the nonlinear inelastic response range). This effectively implies the presence of a time-lag between the moment when the peak value of base-shear is observed and the instant at which the corresponding displacement is recorded; the latter typically arriving with a slight delay with respect to the former. In the definition of the SDOF backbone capacity curves, such time-lag obviously needs to be removed (since it has no physical meaning within a SDOF representation of the response), this being the reason why the black dots in the plots below (representing the max shear-displacement points with the time-lag removed) do not necessarily always appear on top of the hysteretic curves (where the time-lag is instead present). These curves/points have been used to produce the SDOF backbone capacity curves for each structural system, as described in the next section.

Figure 4. Example static pushover analysis (left) and hysteresis loops of the 11 dynamic analyses (right) for two index buildings

3.2 SDOF Models

The points of peak base shear and corresponding attic displacement from each nonlinear dynamic/static analysis have been transformed to equivalent SDOF properties. This data has then been used to produce backbone curves for each index building. These backbone curves, together with a hysteresis model (e.g. Takeda) and springs to represent the stiffness and damping due to foundation flexibility and radiation damping (see Crowley and Pinho 2017 for more details), comprise the SDOF models.

Transformation to an equivalent SDOF system has been undertaken using the transformation methodology presented in Casarotti and Pinho (2007). The transformation factor, \( \Gamma_t \), has been calculated using the results of the analysis that led to the maximum attic displacement (\( \Delta_{\text{max}} \)) without
global collapse. At the time step, $t$, of maximum displacement, the transformation factor $\Gamma_t$ has been calculated as follows:

$$\Gamma_t = \frac{\sum m_i \Phi_{i,t}}{\sum m_i \Phi_{i,t}^2}$$  \hspace{1cm} (1)$$

where $m_i$ is the mass of each floor of the model (noting that the roof mass is added to the attic/top floor), and $\Phi_{i,t}$ are the displacements of all floors normalized by $\Delta_{\text{max}}$. The spectral displacement ($S_d$) is calculated by dividing the attic/top floor displacement by $\Gamma_t$:

$$S_d = \frac{\Delta_{\text{max}}}{\Gamma_t}$$  \hspace{1cm} (2)$$

and the base shear coefficient is estimated by dividing the base shear by the effective mass, $m_{\text{eff}}$, given by Equation 3:

$$m_{\text{eff}} = \sum m_i \Phi_{i,t} \Gamma_t$$  \hspace{1cm} (3)$$

Figure 5 shows an example backbone SDOF curve obtained by transforming the points of peak base shear and corresponding attic displacement from each nonlinear dynamic/static analysis. The reduction of base shear after peak base shear has been defined considering the post-peak hysteretic behaviour of the buildings, whilst the base shear is assumed to be zero when the global collapse capacity is reached (which was obtained for all masonry buildings).

![Figure 5. Example SDOF backbone curve](image)

### 3.3 Nonlinear Dynamic ‘Cloud’ Analysis

For the development of fragility functions, which describe the probability of exceeding a given damage or collapse state under increasing levels of ground shaking intensity, a model for the probabilistic relationship between ground motion intensity and the nonlinear structural response of the SDOF system is needed. The approaches that are commonly used for estimating this probabilistic relationship include the cloud method (Jalayer 2003), the multiple-stripe method (Jalayer 2003) and Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell 2002). The cloud method has been selected for the derivation of fragility functions herein due to its simplicity.
The cloud method is typically applied using an assumption of linear variation in the logarithmic space of the structural response with the intensity measure (IM) and homoscedasticity of the residuals (see e.g. Baker 2007). As this method traditionally does not account for the changing earthquake events that contribute to the hazard at different intensity levels (as can be employed with multiple stripe methods), independence of the selected IM to various other properties of the accelerograms needs to be ensured (so-called “sufficiency” of the IM). Admittedly, it would also be possible to use hazard-dependent record selection together with the cloud method, but given that this would require the output of the seismic hazard model to be available a priori (whereas instead all components of the risk assessment are currently developed in parallel), the simpler approach of using a wide range of records, and ensuring sufficiency, has instead been chosen herein.

The cloud method has hence been employed using a large suite of records to reduce the effort required to select/scale the records that would be appropriate to capture the nonlinear response of each different structural system. Thus, by applying the same large number of records to all SDOF models, a wide range of nonlinear structural response (from pre-yield to collapse) can be captured for all typologies, together with an adequate modelling of the record-to-record variability.

The nonlinear dynamic analyses of each SDOF system have been undertaken in OpenSees (McKenna et al. 2000). Given that the focus is currently on predicting the nonlinear behaviour after structural damage, the response data below 80% of the yield displacement has been removed; the aleatory variability in the pre-yield response is much lower than its post-yield counterpart (and is zero when the same damping is considered in the SDOF system and spectral ordinates), and so removing these points helps to create a set of data that is more likely to be homoscedastic (i.e. with constant variance, regardless of the intensity of the ground motion). Furthermore, the aforementioned assumption of a linear relationship between the logarithm of response and the IM is also more reasonable when the data is focused only on the nonlinear response. Nevertheless, for each SDOF model, it has been checked that the hypotheses of a linear relationship and homoscedasticity were reasonable.

Once the maximum nonlinear dynamic displacement response of a given SDOF (Sd) is obtained from all n ground-motion records, each response (di) is plotted against a scalar/vector intensity measure (IM=[IMi, i=1:m] where m indicates the number of variables that define the vector) and the statistical parameters corresponding to the lognormal distribution of Sd(IM) can be extracted. In particular, the expected value, E[ln Sd(IM)], is modelled by a linear regression equation (Equation 4) with parameters b0 and bi (i=1, . . . ,m), whilst the standard deviation or dispersion (Equation 5) is estimated by the standard error of the regression:

\[
E[\ln D|IM] = \ln d|IM = b_0 + b_1 \ln(IM_1) + b_2 \ln(IM_2) + \cdots + b_m \ln(IM_m) \tag{4}
\]

\[
\beta_{D|IM} \approx \sqrt{\frac{\sum (\ln(d_i) - \ln d|IM(IM_i))^2}{n-(m+1)}} \tag{5}
\]

As mentioned above, the parameters b0 and bi are the estimated regression coefficients obtained by performing a multivariate linear regression. In order to correctly treat the results of the nonlinear dynamic analyses where the displacement response exceeds the expected ultimate displacement capacity (and thus these SDOF systems are deemed to have exceeded the collapse limit state and the estimated displacement response is no longer reliable), a censored regression (Stafford 2008) has been undertaken when estimating the coefficients of Equation 4, as further described in Crowley et al. (2017). An example cloud data plot with censored regression is shown in Figure 6, where the censored observations have been plotted at the limiting displacement capacity value.

Although it is common to check the sufficiency of the IM with respect to magnitude and distance (see e.g. Luco and Cornell 2007), the dependence on a measure of ground shaking duration has also been considered herein, given the evidence from previous studies that the response of unreinforced masonry structures (and other strength and stiffness degrading systems) is dependent on the duration of strong
ground shaking (e.g. Bommer et al. 2004). A number of different scalar and vector intensity measures have been considered for each structural system and the simplest, sufficient intensity measure has been selected in each case.

Figure 6. Example cloud data plot with censored regression

### 3.4 Structural Fragility Functions

The regression analyses described in the Section 3.3 allow equations to be derived that relate the level of shaking with an estimate of the displacement response of an equivalent SDOF system \(S_d\). By identifying the thresholds to damage or collapse in terms of SDOF displacements (or drifts, obtained by dividing the SDOF displacement by the effective height of the SDOF), it is possible to produce fragility functions that describe the probability of exceeding a number of distinct damage/collapse states. The variability in these damage/collapse state thresholds \(\beta_{DL}\) should be accounted for in the dispersion of the response, and can be combined with the record-to-record variability \(\beta_R\) (from the cloud analysis) and an additional building-to-building variability to account for changes in stiffness and strength of the backbone curve across the building class \(\beta_{BB}\):

\[
\beta_s = \sqrt{\beta_R^2 + \beta_{BB}^2 + \beta_{DL}^2}
\]

The damage/collapse state threshold variability has been assumed constant here for the simplification of the risk engine, with a value of 0.3 has been assumed herein, based on studies in the literature (e.g. Dymiotis et al. 1999; Borzi et al. 2008).

The probability of exceeding the limit displacement to each structural damage or collapse state under a given level of ground shaking is calculated as follows:

\[
P_{eDL} = 1 - \Phi \left( \frac{\ln(DL) - \ln n_{SdIM}}{\beta_s} \right)
\]

where \(\ln n_{SdIM}\) is given by Equation (4) and \(\Phi()\) is the cumulative distribution function of the standard normal distribution, DL is the displacement limit of each damage or collapse state (provided in metres), and \(\beta_s\) is the total dispersion (due to record-to-record variability, backbone stiffness and strength variability and damage/collapse state threshold variability).
The values of the displacement limits to damage have been obtained from the experimental test campaign (Graziotti et al. 2017b) and recommended values in the literature (e.g. FEMA 2004). Three damage states, similar to DS2, DS3 and DS4 in the EMS98 damage scale (Grunthal 1998), have been identified for each structural system. For the limit states to collapse, a detailed description of the collapse mechanisms observed in each of the nonlinear dynamic analyses that have been run in either LS-DYNA or ELS, described in Section 3.1, has been produced. The time in the analysis at which the collapse mechanism was initiated has been identified, and the maximum attic displacement up until that point in the analysis has been reported. Up to three collapse states (CS1, CS2 and CS3) have been observed, with the first two being partial collapse states and the final being global collapse (as illustrated in Figure 7).

Figure 7. Collapse states observed in ELS during three different nonlinear dynamic analyses of one of the index buildings

3.5 Chimney Fragility Functions

The fragility functions for chimneys have been taken from the study by Taig and Pickup (2016), who used the observed damage and collapse of chimneys from a number of earthquakes including those of Liege (1983) and Roermond (1992). Taig and Pickup (2016) have proposed empirically-based lower and upper bound step functions in terms of bands of PGA (g) for the probability of collapse of chimneys of buildings constructed before and after 1920 (see Figure 8).

Figure 8. Empirical chimney fragility functions (Taig and Pickup 2006)
4. FATALITY CONSEQUENCE MODEL

The consequence model developed for the Groningen field builds upon the casualty model of Coburn and Spence (2002), which considers a number of factors (M1 to M5) to calculate the number of human casualties (N) in a given building, following ground shaking.

Factors M1 and M2 are used to estimate the number of people within the building at the time of the earthquake, which are not needed for the estimation of inside local personal risk (where a single person is assumed to be permanently located within the building and spread uniformly across the total internal floor area of the building, or with some distribution across the floors where some floors, such as attics, are infrequently accessed) and are instead already accounted for in the exposure model for group risk.

The M3 factor defines the percentage of the occupants that are trapped by collapse and are unable to escape. Coburn and Spence (2002) have estimated average percentages for masonry and reinforced concrete buildings separately, considering the intensity and characteristics of the earthquake. Given that trapped people have to be located within the portion of the structure that collapses, this factor was herein replaced with the probability that the fictional person is trapped, which can be represented by the percentage of total floor area inside the building that is impacted by collapsed debris (given by the area of inside debris divided by the total floor area). This latter percentage is estimated as a function of the collapse mechanism (which could be either partial or complete) and the average floor area of each building typology, and is calculated by combining the collapse observations of the nonlinear dynamic analyses and the footprint area data included in the exposure model.

The M4 factor identifies the percentage of trapped occupants that are killed instantaneously. The percentage of the surviving trapped occupants who subsequently die is given by factor M5, and depends on the building material and the effectiveness of search and rescue (SAR) efforts. Coburn and Spence (2002) propose values for both of these factors for different structural materials.

For outside risk, it has been shown in Taig and Pickup (2016) that the probability of dying when being hit by falling debris outside of a building is close to 1, and hence the probability of dying outside buildings for each collapse mechanism (which could be due to collapse of the structure or small non-structural elements such as chimneys and parapets) is simply calculated from the ratio of the area of debris outside the building (for each collapse mechanism/element) and the outside area at risk.

5. CONCLUDING REMARKS

The fragility and consequence models presented herein build upon an extensive modelling and experimental testing campaign. This paper has described how the outcomes of such studies have been utilised for the development of damage and collapse fragility functions and a fatality model for the most important structural systems that are present in the Groningen field. These functions have then been used in a Monte Carlo risk engine to estimate Local Personal Risk (LPR), group risk (Maatschappelijk Risico) and group damage plots - so-called Maatschappelijk Risico (Schade) - for all buildings in the field.

6. REFERENCES


