Appendix G Review of the report Hazard and Risk Assessment by the SED – November 2015.

The Staatstoezicht der Mijnen requested a review by the Swiss Seismological Survey of the report “Hazard and Risk Assessment – Interim Update November 2015” prepared by NAM in November 2015.

This review was prepared by Stefan Wiemer.

The review report by the Swiss Seismological Survey can be downloaded using the following link:

https://www.sodm.nl/onderwerpen/aardbevingen-groningse-gasveld/aanvullende-informatie-bronnen-bij-advies-sodm-december-2

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Zürich, Nov. 22 2015

Dear [redacted] dear [redacted]

The State supervision of Mines has appointed me in the summer 2015 as an expert reviewer and advisor on induced seismicity in the Groningen area. As part of this mandate I was asked to review the report "*Hazard and Risk Assessment for induced seismicity in Groningen (interim update November 2015)*" by NAM, dated November 7 2015. My review will be based primarily on the aforementioned report; however, it will also consider on the various supporting documents supplied by NAN as well the discussions during the two feedback meetings I attended.

Please note that my review expresses my own personal opinion as a seismologist with expertise on seismic hazard and risk assessment. It does not represent an official position of the Swiss Seismological Service or ETH Zurich. Please also note that, in compliance with your request, my review will be mostly focused on the 'big picture', not on detailed questions on individual subcomponents of the model or computations. While there are numerous suggestions in my review, I am well aware that some, or even the majority, will end up not being implemented. Opinions differ and overriding constraints may exist. Please consider my ten recommendations as suggestions from a somewhat outside perspective, which hopefully you will find helpful input for your difficult task. Finally, reviewers are by the very nature of their assignment often critical. I think it is important to state that overall I observe great and rapid progress made the past three years towards an quantitative assessment and management of the risk posed by induced earthquakes in the Groningen area.

I would agree if my review, or parts of it, are made available to other scientist or even publically if you consider this appropriate. In this case, I would appreciate to be informed beforehand.

With kind regards

Stefan Wiemer

1. Comments on the overall approach and current status

The dramatic increase in induced seismicity in the Groningen area poses a problem of great societal and economical relevance. In such situations, it is the role of science and engineering to produce the factual baseline for discussion and decision-making. Ideally, risk-cost-benefit analysis offers a transparent pathway to assemble and integrate relevant evidence to support such complex decision-making, considering also the uncertainties and knowledge gaps. In a recent review article, Fischhoff[1] outlined and discussed in some depth this kind of approach, based on selected case studies.

I fully support the path adopted by the operators (in short NAM from hereon) and State supervision of mines (SodM) to build up a fully probabilistic, state of the art (and partially beyond), time-dependent seismic hazard and risk model. The model that is now emerging, if successfully calibrated and partially validated, should in the near future enable risk analysis, risk assessment and risk management, including decision support for control and mitigation actions.

I also overall support the methodology adopted, which in essence decomposes the complex overall system into manageable components (the well-known bridge or train) and then calculating the overall system response. The model largely follows also the established best practice of representing uncertainties in formalized ways, although, as detailed below, there is a heterogeneous, partially inconsistent approach adopted across the different components of the overall model.

The operator invested substantial and much appreciated efforts into gathering over the past two years a wealth of geophysical, geotechnical and engineering data of relevance. These data are essential to calibrate the model components for a site-specific Groningen model, and the efforts are nicely starting to find their way into the model. In my assessment detailed below, the work has progressed at somewhat different speeds and with different rigor in the various model components, specifically, the seismological analysis and model building is somewhat lagging behind.

Recommendation 1: The overall approach adopted by the operator to the assessment of the risk posed by induced earthquakes in the Groningen area is, in my opinion, fully appropriate and almost without alternative. It should be continued and extended in the future, in an evolutionary sense.

The model version 2.0 as of November 2015 is clearly much more advanced and appropriate for decision making than version 1.0. The methodological approach has been refined, vast amounts of new data have been integrated, various sensitivities tested, etc. The complexity of the model and the number of (free) model parameters has also greatly increased. As a consequence, model verification, calibration and independent validation is becoming increasingly a challenge, one that is in my assessment only partially addressed so far. Also, the interdependencies between model components have not been fully explored. It is a well-known fact in the re-insurance business that combining all the best individual component of a complex risk model may in the end lead to a model that is in contradiction to the historical record of shaking and damages. Issues that have not yet been explored in many depths, for example, are the potential of double-counting uncertainties between ground motion, site amplification and building fragilities.

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[1] B. Fischhoff, Science 350, aaa6516 (2015). DOI: 10.1126/science.aaa6516

It is also a fact that much of the model and model calibration has been done under great time pressure, and that some of the model components are less well tested and developed than others, as detailed below. The model is clearly at this stage 'work in progress', and will likely continue to evolve substantially in the next months. Not all model parts and decisions taken along the way are fully transparent and reproducible, and no independent verification of model components has take place. There is a certain degree of arbitrariness in decisions on parts of the logic tree model adopted, and on the weights assigned, which could potentially have a large impact on the overall hazard and risk levels.

Recommendation 2: The model in its current development stage is in my assessment not yet robust enough for drawing firm quantitative conclusions about the absolute level of hazard and risk, its spatial distribution and the effectiveness of risk reduction strategies. However, it is conceivable the model version 3.0 will have evolved such that informed decisions on the Winningplan 2016 are feasible.

2. Comments on process ownership and risk governance

Successful risk governance is an analytical-deliberative process in which field operators, independent risk analysts, regulators and stakeholders collaborate in managing risks. Key elements of this process are a clear separation of roles, the transparency of the process and the quality of the exchanges and communication. Ultimately, the acceptance of the risk assessment for decision-making transpires from the trust in the models and in the ones that have constructed the model. In the Groningen induced seismicity case, trust into the field operators has at least been partially been lost, and re-gaining trust inevitably will be a substantial effort and take a long time.

In the seismic risk assessment of Groningen, there is also in my assessment a potentially detrimental conflict of interest between the interest of NAM in operating the gas field ways that maximizes the economical gain and the recommendations from the seismic risk model on minimizing the risk. The potential impact of this conflict of interest is strongly amplified by the existing mistrust between the various parties involved. Added to this mix is the fact that the risk model is highly complex and computationally demanding, with countless model parameters, and numerous decisions to be made in all stages of the modeling development, verification, calibration and validation. Currently, it is impossible for SodM, TNO or KNMI to fully verify or reproduce the results of the computations risk; even understanding exactly what has been done and what the model sensitivities are is a challenge in such a complex and rapidly developing model.

In my assessment, based also on the experience of similar entrenched discussions on shale gas in the UK, and nuclear energy in Switzerland, it is well possible that the risk model will not be widely accepted as a basis for decision making because of this - perceived or real - conflict of interest, the complexity and irreproducibility of the model, and the partially unclear definition of roles of individuals and organizations. An additional layer of complexity is added by the fact that (1) many of the staff members have been employed at various stages in their careers by the various organizations (NAM, SHELL, KNMI, TNO, SodM), that (2) much of the primary data is collected by KNMI and TNO and that (3) the roles of external experts used by the project, by the external consultants and by state agencies are not always clearly defined. There are clear benefits to the collaborative process adopted in many risk governance projects. Structured feedback, and open discussions between all parties are vital components of a successful project. However, these need to be embedded in a framework that transparently establishes well-defined roles for all parties.

In my assessment, the acceptance of the Groningen seismic risk model would ultimately be much higher if the model and the process to build, maintain and update it would be owned, or at least co-owned, by an agency that has no obvious conflict of interest. For example, through a long-term contract the field operator, regulator and possibly other parties would all have full access to and co-ownership of the model and data, the model would be developed in a collaborative spirit. The partners would oversee and co-own the development roadmap of the model and guarantee its continuous review and quality assurance. External consultants conduct much of the work in any case, and this could be continued. However, the consultants should then report to an independent agency rather than NAM.

Recommendation 3: The process of risk governance overall should be reflected on beyond the technical aspects of risk assessment, in order to maximize the wider legitimacy of the work. In the current setup, the roles and interactions of involved parties, but also the ownership of the model overall, is in my view problematic.

3. Comments on the use of consultants, external experts for review and quality assurance

NAM is relying on a number of external experts to build the risk models. In my assessment this is a key element of the quality assurance and ultimate success of model building. NAM has attracted some of the leading experts and consultants in the field of ground motion prediction, local site response, geotechnical aspects and earthquake engineering. These experts have extensive experience in project management of similar projects, a further key ingredient for success. They seem to have established a good working relationship with staff at NAM. The one exception to this is the seismic source model (from gas production to hazard, chapter 3), where, to my knowledge, in-house experts conduct most of the developments.

As stated in the report in various places, and listed also in Appendix B of the report, a large group external experts has been involved in various stages of the process and for selected components of the modeling. This kind of external expert involvement is potentially highly useful to improve the quality of the work. However, I have three concerns related to the involvement of external experts:

- The use of experts is somewhat inconstant between the various components of the model. The GMPE and earthquake engineering on the one hand parts have established a somewhat formalized hazard and building fragility review team, and use them to a certain extend to capture also the epistemic uncertainties and to assign logic tree weights. However, the 'seismogenic source model' and - partially - the site characterization, seems to involve external experts to a smaller degree, and in a less formalized way.
- There is an apparent lack of clear rules and responsibilities of external experts, observers, guest and reviewers. Appendix B for example is a long list of people that had at some point interactions with the project (including myself), with no distinction and responsibility. The written reviews of external experts should be made openly available, and the way the expert feedback is used, or not used, justified. It is not clear if the feedback workshops are seen as expert elicitations, if they reflect also on 'ownership' of the model and how they specifically help to capture uncertainties.
- The choice of the experts is somewhat arbitrary. At least in theory it is possible that this choice biases the feedback and input received.

Recommendation 4: The use and roles of external experts should in the future be more formalized and applied consistently across the various model components. More expert feedback/elicitation especially on the 'sources' model seems appropriate.

4. Comments on the analysis of seismic data

A key element for enhancing process understanding and forecasting skill in the future is in my assessment a more advanced analysis of the micro-seismicity. In my opinion[2], there are numerous first-order questions of relevance that need to be addressed in much more detail than attempted so far. I list a few examples below:

1) Improving the precision and accuracy of hypocenters. Using advanced (re-) location techniques, applied to past data as well as in near-real time, this will also require a new minimum 1D and 3D seismic velocity model (P- and S-; separately) as well applying relative re-location algorithms. Potential benefits include:
    a. Improved spatial distribution of patterns of seismicity as input for rate density models, characterizing clustering etc.
    b. Ability to correlate faults and hypocenters, leading to an improved geo-mechanical understanding of the coupling between compaction, faults and seismicity.
    c. Ability to analyze with more confidence migration path of seismicity, identification of lineaments etc.
    d. Improved depth resolution, ability to detect activation of basement faults.
2) Targeted efforts to detect and locate smaller earthquakes, using for example automated template-matching approaches. Potential benefits include:
    a. Enhance the existing and future seismicity record by an order of magnitude in terms of the number of events, leading to much more robust statistical patterns (rate density, size distribution) and geo-mechanical interpretations.
    b. Improved ability to calibrate and validate forecast models.
3) A Groningen specific space-time model of completeness and homogeneity of magnitude reporting (using Gutenberg Richter based techniques, BMC, or PMC combined with noise analysis at the sites). Given the rapid changes in the network, such a space-time model of the reporting completeness will be important to be able to assess the relative earthquake size distribution, defined the activity rates with high resolution but also help in the optimization of the network performance.
4) Improved source characterization of future and past events, integrating also the local site amplifications, in order to determine in near-real time also moment magnitudes, corner frequencies and stress drops for small events, and revisit old magnitude and magnitude scales. This will lead to:
    a. Improved assessment of space-time patterns in the earthquake size distribution.
    b. Improved link of physical parameters to ground motion predictions (e.g., stress drop of events as a function of location,).

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[2] There is an obvious potential bias in my stated opinion that the analysis of seismicity data and the seismogenic source model are most in need of additional work: It reflects my core competence and in other domains my own ignorance may prevent me from seeing the need for additional work.

5) Improved ability to assess focal mechanism of also smaller events.

    a. Improved link between faults and earthquakes, input for the geo-mechanical model.

6) Systematic search for slow/unusual events or non-volcanic tremor, possibly an indication of a-seismic motions, and potentially linked to the partitioning between seismic and a-seismic deformation, a poorly understood feature of the deformation.

7) Precursory activities as part of warning. Using the improved seismicity is may be possible to detect trends in micro-seismicity that could be indicative of upcoming larger (M>3) events. This may have potential for short-term (days to weeks) hazard and risk assessment.

This list is not very original; most of the proposed methods are based on well-established techniques documented in the literature. Much of this work is as far as I know already on the way or anticipated in the future, and the importance of addressing these questions is generally well established. In my assessment, the limits of the seismological analysis are the primary bottleneck limiting the development and validation of forecasting models.

There has also been a commendable effort to improve the seismic network in the Groningen area to an adequate monitoring level. In my assessment, however, the progress in addressed the aforementioned questions is too slow, given the urgency of the needs of the hazard and risk model. It is also substantially slower than progress in the GMPE/site and building parts of the model. This may be partially related to the fact that considerably fewer resources are available for seismological analysis. External consultants have been hired by NAM for GMPE and building parts, whereas the R&D on seismicity analysis resides largely with KNMI with less flexibility to substantially scale up the effort rapidly, in addition to the network building and hazard modeling also ongoing at KNMI.

Recommendation 5: The efforts related to analyzing and interpreting the seismological data should be prioritized and up-scaled substantially. This may require in addition efforts of groups outside of KNMI and NAM.

5. Comments on the seismogenic source model

The seismogenic source model is by requirement the most innovative part of the entire analysis chain. In the GMPE, site and building parts, the work is ambitious in scope but in essence based on existing state of the art and methodology. However, there is no precedence for such a complex time-dependent seismogenic source model that links production rates and future seismicity. The implications of the rate model is also the one that is most understandable to non-scientist, since the magnitude and frequency of events is easily understood and checking the model against observations is straightforward.

The seismogenic source model as it stands now is in many ways innovative and sophisticated; however, I question that it covers the requirement for PSHA to be exhaustive in capturing the epistemic and aleatory uncertainties. A key requirement of a PSHA is *"to represent the center, the body, and the range that the larger informed technical community would have if they were to conduct the study"*.

The current model implements one possible pathway to a rate forecast, and calibrates it to observations, achieving a good fit. Alternative implementations using, for example, largely statistical models, using rate and state approaches, using strain-rates rather than compaction and strain-thickness, using different parameterization of b-values, and accounting differently for the potential of 'non-linear', unexpected behavior of earthquakes could be imagined and are documented in the literature on induced earthquakes. These alternative models may all fit the observed rates equally well, but they may substantially differ in their forecasts and in the impact that production changes may have on future seismicity.

In addition, the seismogenic source model in its current state of documentation no fully reproducible and transparent. The compaction model, as well as the coupling of compaction to rates, is described not in sufficient detail in the available documentation, or only in somewhat outdated versions were not all recent decisions takes are well documented and well justified.

The spatial and temporal variability of b-values is clearly a driver of the risk, and its future evolution one of the key elements that could substantially alter the hazard and risk estimates. This is also a substantial source of uncertainty, given the limited physical understanding of the link between b and stress/strain/strength/structure, and given the limited data available in Groningen. Coupling the strain thickness to b-values as applied in Version 2.0 is an interesting idea, but has to my knowledge not been done in past hazard studies nor published in the peer reviewed literature. A number of alternative models exist (dependence of b-values on differential stresses, on faulting styles, only on space-time patterns, constant overall, etc.), a source of uncertainty that has in my assessment not been fully appreciated in Model 2.0.

I suspect that hazard and risk sensitivity estimates show currently a weak dependency on the seismogenic model also because the uncertainties have not been explored as systematically as done for the GMPE, site and building part. This is especially important since it has the potential to also underestimate the effect of changes in production rate on earthquake hazard and risk.

Recommendation 6: The seismogenic source model is in my assessment not diverse enough to satisfy the usual PSHA requirement of *capturing the center, body and range of the informed technical community*. Additional efforts are warranted to widen the model/uncertainty space.

The potential for using ensemble models

Ensemble models are a well-established tool in many areas of sciences, such as weather and climate forecasting. Recently, such methods have been adopted to include time-varying hazard and risk assessment of natural seismicity as well as induced seismicity in geothermal contexts. In all of these cases, ensemble models can indeed outperform each individual model, especially if the model weights are adjusted dynamically, based on the past performance of the entire model, or of model components. The Bayesian framework implemented by NAM for Model version 2.0 likely is similar in spirit; however, it appears to be used primarily to select models, not combine them. Ensemble forecast framework works best if a wider range of input models is used to make forecasts, and if it is used also in near-real time to update forecasts and weights on the fly as new data arrives and model performance is re-assessed automatically. A dynamically weighted ensemble model of a rage of models, some simple, some complex, would in essence replace the need for a logic tree to capture uncertainty in the time-dependent forecasting model. It would likely result in a more robust forecasting model overall.

Recommendation 7: The use and potential benefit of integrating ensemble models should be further explored and formalized.

Model validation using forecasting experiments

The principal challenge to the entire model, but specifically to the time-dependent seismogenic source model, is in my assessment the validation of the model against independent data. This is a common problem in many areas of science: Extrapolating a model outside of the range calibrated by data is challenging, and it is then when over-parameterization can make a big difference and where more simple models may in fact perform better than complex ones. In the case of Groningen, extrapolations into the future and outside of the magnitude range experienced are the key challenges for models.

In my assessment, a critical element of model validation and ultimately of building up trust in the model by scientists and the public would be a somewhat formalized, community accepted approach to test the model performance against data. This includes:

- The formalized use of pseudo-prospective test: how well do different models forecast the space-time-magnitude evolution of observed seismicity when given a limited learning period?

- The use of fully prospective, formalized and potentially independently conducted assessment of the forecast performance. This gold standard in testing is widely applied also in the medical industry: Any drug must be proven to work in so-called double-blind studies. The same standard could be applied here.

Formalized testing is first of all a powerful tool to analyze and understand the performance of models: Where and when do they do well, when do they fail? How well can we forecast future seismicity? There is a wide set of literature in the seismological community on forecast model evaluation, which is increasingly applied when evaluating especially time-depended forecasting models and 'Operational Earthquake Forecasting'. Much of this has been conducted in the spirit of the Collaboratory of Earthquake Predictability (CSEP; www.cseptesting.org). There are also applications to induced seismicity emerging in the literature.

Recommendation 8: Formalized, independently conducted testing of the future performance of seismicity forecast models should be considered as a key ingredient to improve model building, an element of model validation and an important component to build up confidence in the performance and reliability of the seismogenic source model.

6. Comments on the Ground Motions Prediction Model

One oft he major achievements of model 2.0 is clearly the development of a Groningen specific ground motion prediction model (in short 'GMPE model'). There has been a substantial investment of resources, resulting in substantial progress. Given the importance of the GMPE mode, reconfirmed in sensitivity analysis, these efforts are well justified. The work is conducted by some of leading scientists in the field and reviewed by an impressive group of international experts.

The key uncertainty in this domain is extrapolating to the ground motions of large magnitudes, which have not been observed in the instrumental record so far. To capture this uncertainty for the rock hazard, a logic tree with a number of stress drop values is adopted.

Key questions that in my view should be addressed with even more care in Version 3.0 of the GMPE model are:

- What is the evidence and how certain can one be that also induced earthquakes will be of lower stress drop, even if they would not fall with the reservoir and considering the fact that mostly data from injection (not depletion) related induced seismicity may (in some cases) have lower stress drops?

- Can it be already fully justified to only use a stochastic model, ignoring empirical GMPEs?
- Because designing and setting the weights of the logic tree for different stress drops has a large effect on hazard values, the process of defining branches and setting weights must be as transparent and independent as possible. How can this be done more independently and more formalized? Who is in the end responsible for setting the weights? What is the role of the international experts in assigning the branches and weights?
- Are there alternative ways to translate rock hazard to local site conditions, is the current approach capturing the considerable uncertainty well enough?

Two of these concerns related back to process of risk governance overall, and on the use of experts (my points 2 and 3).

Recommendation 9: Recognizing the importance of the GMPE logic tree and its weight to the overall hazard level, designing of the tree and setting these weights must be achieved as transparently and independently as feasible. The benefits of structured expert elicitation should be considered.

7. Comments on the Maximum Magnitude Assessment

Model Version 2.0 adopts a simple logic tree to express the large uncertainty of the maximum magnitude possible. This is a reasonable intermediate approach that more accurately reflects the fact that indeed there is a large uncertainty and widely varying opinions. It is also sensible that a workshop on the subject will be conducted in the preparation of Version 3.0. However, in my assessment there are potential problems in the procedure adopted:

- To what extend is it possible in a short workshop of experts only partially familiar with the regional context and with little familiarity to the Groningen seismogenic source model to draw meaningful conclusions and distill them into a logic tree?
- What is the ownership these experts will have on the Mmax model and how will their opinions be integrated in a formalized way to set the weights of an Mmax logic tree? How and by whom are the experts selected?
- Is it feasible to decouple the Mmax problem from the rest of the seismogenic source model, where this group of external experts has no involvement and ownership?
- Could there be useful preparatory work on Mmax estimates that will enrich/collect the empirical databases, present selected case studies or dedicated attempt to modeling? Do, for example, the extreme case such as the three Gasli M7 earthquakes in Uzbekistan, or the M5.4 Lorca earthquakes in Spain, offer meaningful insights?
- Can the potential of the seismicity to be triggered beyond the reservoir area be quantified, such as the potential to re-active faults in the basement?

Recommendation 10: The Mmax workshop is likely to be focused on highly controversial topics with widely varying opinions between exerts. It is important to prepare the workshop well, also considering if a more structured expert elicitation is needed or useful.

8. Comments assessing building fragilities

The work on characterizing the fragilities of buildings has in my assessment, greatly advanced in Version 2.0, owing to the considerable and well-directed efforts of the earthquake-engineering domain. Not being much of an expert in the domain myself, I am impressed by the efforts undertaken, which should clearly aid in quantifying the risk and target the retrofitting efforts. The role of external experts to review the model is also much appreciated for quality assurance, although my comments on the role and ownership of experts apply here also.

The only concern that I would like to comment on is an obvious one: Does the current fragility model capture the diversity of buildings? Are for example the impressive full-scale tests conducted in Pavia indeed representatives for this building typology, are the effects of imperfections, aging etc. well enough understood to draw firm conclusions? Are there additional validation experiments in the field, or numerical modeling, that can firm up the transferability of the results to the wider Groningen building stock?

9. Knowledge transfer from other fields

There are countless oil and gas fields on Earth that are experiencing subsidence similar to the Groningen area. Groningen is somewhat unique not because of the induced seismicity observed but because of the population density. However, there is only very limited information available on seismic events in these fields, because the relevant data does not exist or is not shared openly by operators and regulators. In my assessment, much could be learned if more case studies of high quality could be conducted. Because Shell and Exxon are major players in oil and gas extraction, they may be in a position to advocate the benefits of more transparency on induced earthquakes for all involved parties, and develop a strategy how validation of models can also be advanced by application to a wider range of case studies.

Appendix H Review of the report Hazard and Risk Assessment by the USGS – November 2015.

The Staatstoezicht der Mijnen requested a review by the US Geological Survey of the report "Hazard and Risk Assessment – Interim Update November 2015" prepared by NAM in November 2015.

This review was prepared by Art McGarr and Bill Ellsworth.

The review report by the US Geological Survey (USGS) can be downloaded using the following link:

https://www.sodm.nl/onderwerpen/aardbevingen-groningse-gasveld/aanvullende-informatie-bronnen-bij-advies-sodm-december-2

Review of "Hazard and Risk Assessment for Induced Seismicity in Groningen – Update 7th November 2015"

by

William L. Ellsworth and Arthur F. McGarr
U. S. Geological Survey

Introduction The U. S. Geological Survey (USGS) provides technical review and advice to the State Supervision of Mines of the Netherlands (SodM) under a Letter of Agreement NL-02.0000 dated June 25, 2015. At the request of the SodM, USGS has been asked to review of "Risk Assessment for Induced Seismicity Groningen – Update 7th November 2015." This document is the review. The conclusions contained herein are solely the views of the authors and do not constitute an official position of the USGS or the U. S. Government.

Background This report presents a comprehensive evaluation of the earthquake hazard and earthquake risk posed by ongoing gas field operations in the Groningen region. The report differentiates between hazard, a source of potential danger or harm, and risk, the chance of suffering loss or harm. The earthquake hazard evaluation is based on well-established principles and methodologies of Probabilistic Seismic Hazard Analysis. The PSHA model is built from three main components: 1) the earthquake source model that describes the location and magnitudes of earthquakes in space; 2) the earthquake rate model that describes the rate of occurrence of earthquakes of different magnitudes for each location in the earthquake source model; and 3) ground motion prediction equations (GMPE) that describe the distribution of shaking expected for earthquakes as a function of magnitude, distance (and other parameters). Risk is the product of the hazard with the buildings or other structures exposed to the hazard and their fragility to earthquake shaking. Because our expertise is in the area of earthquake hazard, this review focuses on the hazard, with only a few comments on the risk sections of the report.

The main conclusion regarding hazard, summarized on p. 6 of the report is:

> Hazard maps indicate a smaller geographical area is exposed to significant (> 0.25g PGA) ground accelerations for 2016 – 2021 than was projected for the same period in the May 2015 PHRA report. The reduced hazard area is consistent with the KNMI Hazard map update published in October 2015 and now reflects the improved methodology used to predict ground motion, based on the detailed description of the soil layers in the Groningen field area.

As described on p. 7 of the report, the updated hazard model is based on a revised seismic source model that correlates the earthquake production rate with the production and compaction history of the reservoir; and the development of GMPEs that explicitly account for the local geologic characteristics of the Groningen region.

Earthquake Source and Rate Model The revised seismic source and activity model (Bourne and Oates, 2015a and 2015b) combined the earthquake history of the Groningen field with the subsidence history to develop a forecast model for future activity. It replaces an earlier model that assumed that seismic activity is proportional to reservoir compaction (Bourne and Oates, 2014) with a more fully developed geo-mechanical model. Several key elements of the model include the use of surface subsidence to estimate strain in the reservoir using a thin sheet model, development of a nucleation rate model of seismic events as a non-linear function of compaction rate, and the incorporation of an Epidemic Type Aftershock Sequence (ETAS) model into the framework.

The new model was independently reviewed by Ian Main (Appendix D). We are largely in agreement with his review. As Main points out, this is a novel model that has been calibrated to existing data. Assessing its performance prospectively should be a high priority in the future, particularly if this can be done for shorter time intervals than annual forecasts. Perhaps this will be possible if the improved seismic network reduces the magnitude of completeness.

The correlation between seismicity and compaction, while compelling, does not by itself justify the conclusion that the strain is being partitioned between dominantly aseismic deformation and brittle failure. The exponential relation between compaction and seismicity rate might reflect increasing shear stress within the reservoir. In this regard, it is surprising to us that apparently little has been done to measure the orientation and magnitude of the stress. This would seem to us to be a key component of a comprehensive geo-mechanically-based earthquake source model. What we do know about the state of stress from regional data shows that the north-northwest striking normal faults that cut the reservoir are optimally oriented for slip if the stresses are high enough. What we don't know is if any of the faults are critically stressed.

Improved earthquake detection and location may also provide critical information needed to identify seismically-activated faults and their relation to pre-existing structures. This is vital work. Association of seismicity with faults that extend downward into the carboniferous would raise concerns in our minds that rupture could extend below the reservoir, substantially increasing the maximum possible magnitudes of induced earthquakes. While the work on hypocenter determination by Pickering (2015) indicates that much of the seismicity occurs in the reservoir, in contrast to earlier work, it does not demonstrate that ruptures have been or will be confined to the reservoir.

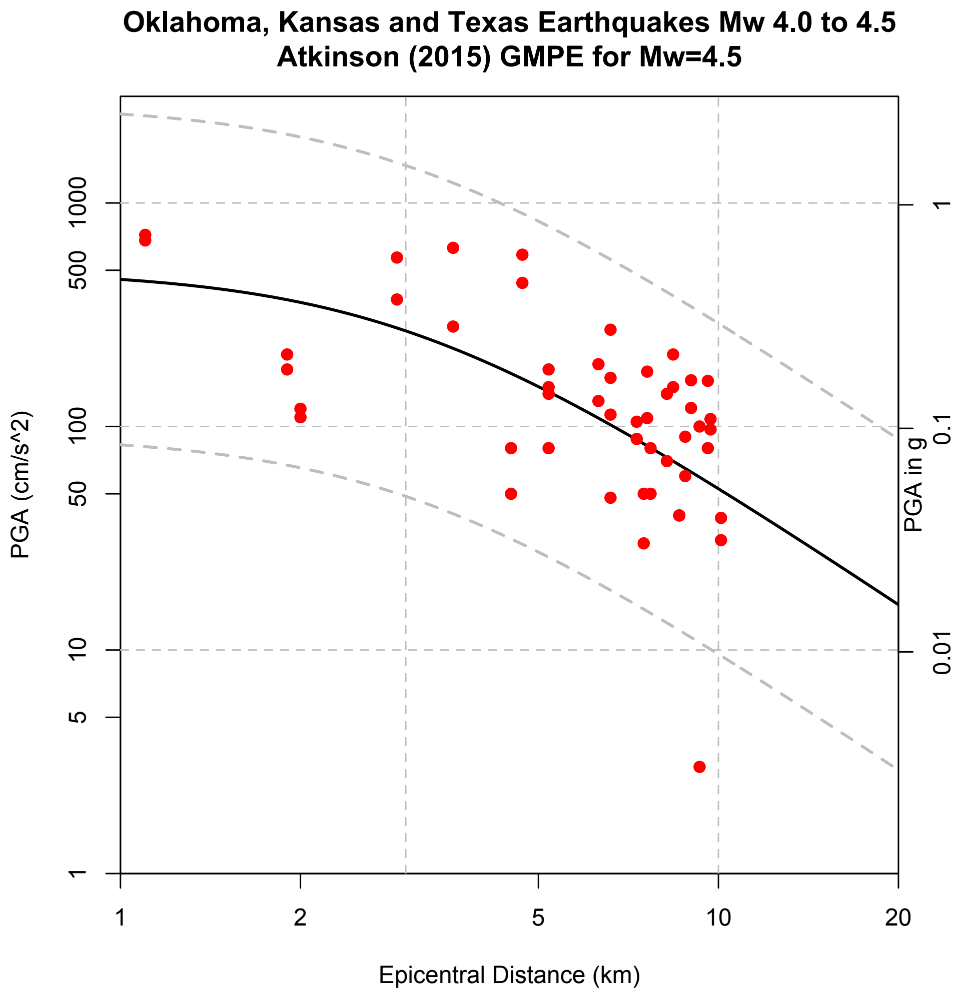
<u>Version 2 Ground Motion Prediction Equations</u> The report by Bommer et al. (2015) "Development of Version 2 GMPEs for Response Spectral Accelerations and Significant Durations from Induced Earthquakes in the Groningen Field" represents a comprehensive body of work that delves deeply into the problem of developing ground motion equations (GMPEs) for unobserved earthquakes in the unusual setting of the Groningen region. The very soft surficial deposits in the area pose a particularly challenging problem for GMPEs, as they are likely subject to nonlinear behavior in strong shaking. Consequently, Bommer et al. (2015) developed equations for ground motions at the top of the competent sediments ("reference rock horizon") and then applied a nonlinear formulation by Darendeli (2001) that accounts for modulus reduction and damping at high strain levels to determine the ground motions at the surface.

The work incorporates an extensive suite of geophysical and geotechnical measurements into the development of both the reference rock ground motions and the spatially detailed site amplification functions. The resulting model is as detailed as any that we are familiar with and represents a significant step forward in developing a comprehensive, geologically-based, site-specific GMPE. Considerable attention is paid throughout the development of the model to uncertainty, ultimately needed in the PSHA to capture the epistemic uncertainty in hazard. As with any model of this complexity, there will be an ongoing need to test its predictions against prospective data, as they become available.

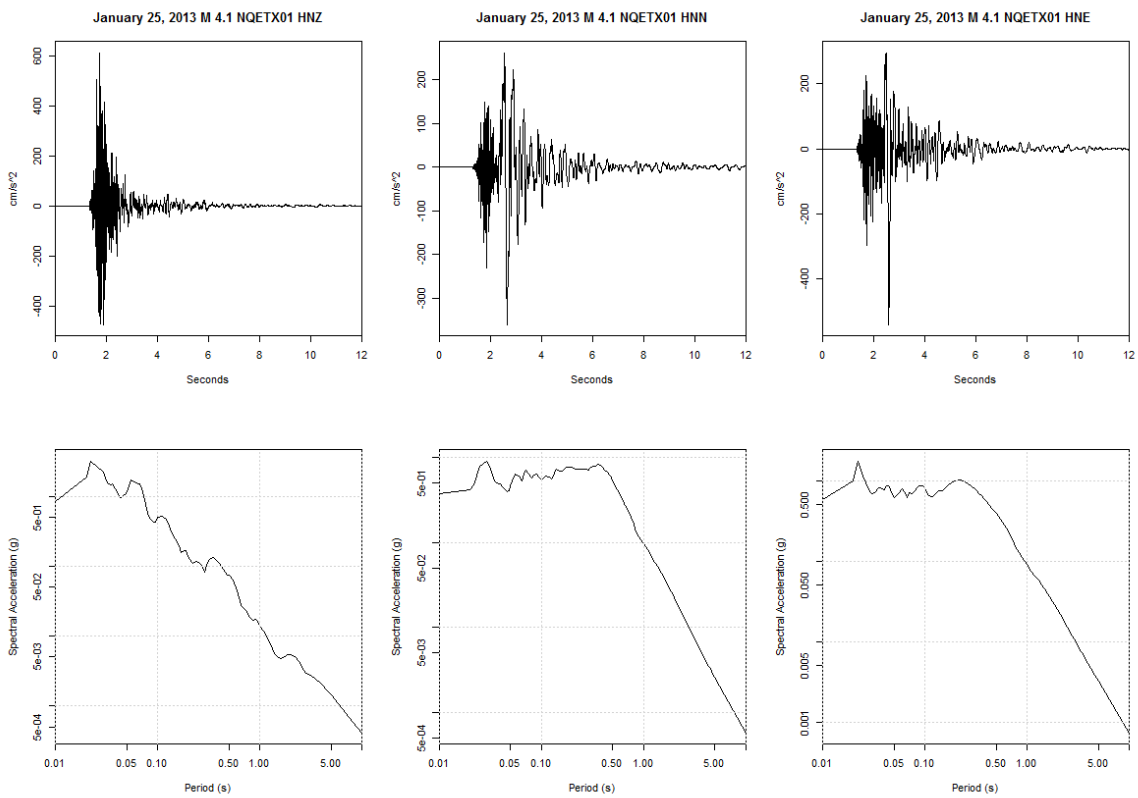
We first discuss the reference rock ground motions. It is scientifically challenging to predict the shaking from earthquakes that are significantly different from those in the existing database. The approach taken here uses theoretical models of earthquakes to synthesize ground motions. The physics of wave propagation is well understood, as is the radiation of seismic waves by the earthquake source. Successful prediction of ground motions thus depends on knowledge of the Earth structure and seismic source processes. At close epicentral distances for shallow earthquakes, wave propagation effects are straightforward, leaving characterization of the average properties of the rupture process the primary unknown. The extensive work on the velocity structure and the attenuation structure from source to top of the engineering rock should be sufficient for accurate modeling of ground motion, given the appropriate source model.

The approach taken here models the source process using a point source approximation. We note that this method is far from state of the art. A stress parameter (equivalent to static stress drop) is used to set the corner frequency in the source model. Stress drop is difficult to measure, as the dispersion of measured stress parameters for the V1 and V2 models as a function of magnitude illustrates (Figure 6.3 of Bommer, et al., 2015). More information about how stress drop was measured would have been helpful for evaluating the results. The low stress parameter model (10 bars) appears to be only marginally consistent with the data for M 3 and above. Low stress drop values often reflect lack of bandwidth in data, which is suggested by the overall trend of increasing stress drop with magnitude and hence greater bandwidth as the corner frequency moves to lower frequency.

As with the V1 GMPEs we reviewed earlier, the resulting reference ground motions surprise us as being rather modest for earthquakes with magnitudes in the range of interest at short epicentral distances. The figure below compares recorded geometric mean peak ground acceleration (PGA) values with the recent GMPE for induced earthquakes proposed by Atkinson (2015). This GMPE curve is for Mw=4.5 and a focal depth of 3 km. The 2.5% and 97.5% confidence bounds are also shown. The earthquake magnitudes are all between Mw 4.0 and 4.5 and have focal depths between 3 and 5 km.



An example of the seismograms and response spectra (5% damping) is shown below for a Mw 4.1 earthquake that occurred at 3 km depth in east Texas. The recording was made very close to the epicenter (~ 1 km). The wave path from the hypocenter to surface traverses a thick stack of carbonates, anhydrites and salt before encountering soft sediments in the upper hundred meters.



As seen in the above two figures, peak ground accelerations, or, equivalently, spectral accelerations at 0.01 s, in the central U.S. are typically about 0.5 g.

For comparison, we see that the response spectral ordinates at 0.01 s period in Figure 6.43 (M 4.5 at 0 km distance) or Figure 6.45 (M 5.0 at 5 km distance) from Bommer et al. (2015) (reproduced below) are at about 0.1 g, or lower.

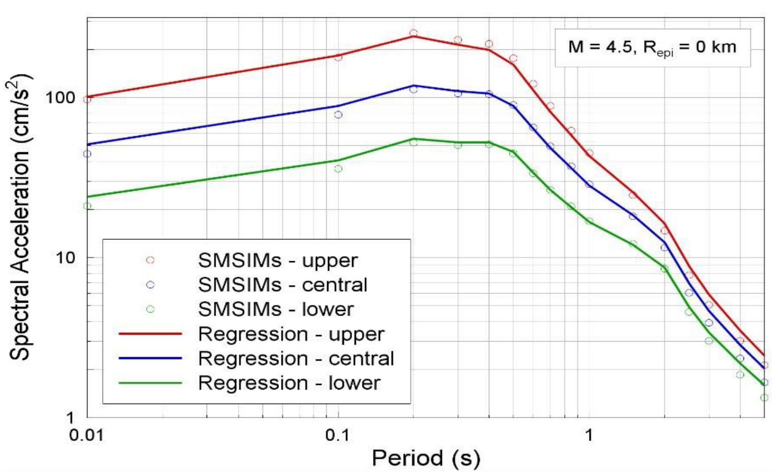


Figure 6.43. Comparison of simulated and predicted response spectra at NU_B due to a M 4.5 earthquake at an epicentral distance of 0 km.

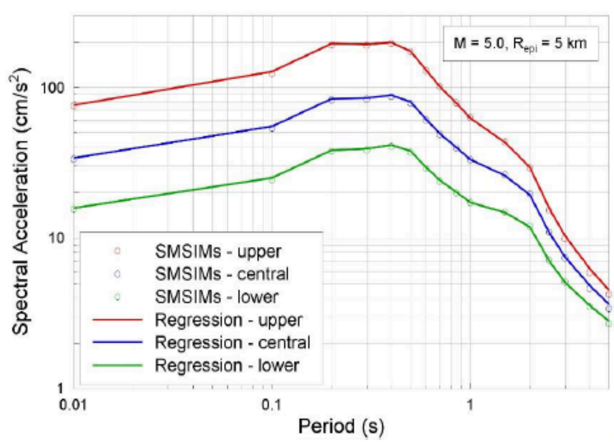


Figure 6.45. Comparison of simulated and predicted response spectra at NU_B due to a magnitude M 5.0 earthquake at an epicentral distance of 5 km

Thus, there is a substantial difference between ground motion parameters for earthquakes induced in the central U.S. and those in the Groningen field. The observed values from induced earthquakes in the central U.S. all exceed the central model and most also exceed the upper model response spectrum. It can also be seen that the observed response spectra for the Texas earthquake above exceed the upper model at all periods (even if divided by 2 to approximately account for the free-surface effect).

We do not claim that induced earthquakes in the central U.S. have the same source spectra as earthquakes of comparable magnitude that might someday occur in the Groningen field. But we can find no valid reason for rejecting them out of hand either. Consequently we caution that when model epistemic uncertainty bounds are inconsistent with data for earthquakes that are nominally similar, but induced by other processes, it is important to understand why this is the case. This question is central to the discussion, as the evaluation of risk is dependent on the hazard model. Perhaps the internal review committee that met in London on October 27-28, 2015 discussed this topic?

Site response. The micro-zonation necessary for assessing risk depends heavily on site response, which, in turn, is a function of the shallow velocity structure. Surface deposits in the Groningen region are highly variable, but generally characterized by soft soils and young geologic deposits that can be expected to have a significant effect on wave propagation through them. To account for the nonlinear behavior of these materials, the theory of Darendeli (2001) has been used. Chapters 7-9 of Bommer, et al. (2015) describe the development of the site response model, site response analysis and site amplification factors, respectively. Although we are impressed with the comprehensive approach taken by Bommer et al. (2015) in accounting for the effects of site response on ground motion at the surface, we are not sufficiently specialized in this area to be critical of their analysis. Although, from our perspective, their results seem reasonable, it might be worth considering the possibility of having an expert, such as Jonathan Stewart, assess this material.

From Hazard to Risk and Probabilistic Risk Assessment Chapters 5 and 6 of the report cover the extensive work being done to identify vulnerabilities to future earthquakes and steps that can be taken to reduce the risk. This is a very impressive effort, balancing collection of instrumental data in structures with engineering testing of representative building types and construction methods. We are equally impressed with the survey results of potential hazards, with on the order of 150,000 buildings surveyed used to develop the detailed exposure database.

Brief summary. The V2 GMPEs reduce the ground motions and consequently both the hazard and risk are lower than previously believed. Moreover, the GMPSs give ground motion parameters that are significantly lower than found for comparable magnitude earthquakes elsewhere. It is important to understand why the ground motion models for the Groningen field are so much lower than for their counterparts in the central U.S. The questions we have asked about the new GMPEs suggests to us the need for additional research to improve the model, especially if it turns out that much larger magnitude earthquakes may need to be taken into consideration. If so, then it may be necessary to employ state-of-the art methods for synthesizing finite ruptures in place of point source models. Disaggregation of the hazard identified moderate magnitude events (M 4-5) as the primary contributor in the area of greatest hazard (Loppersum), but larger events pose a greater hazard to the city of Groningen. If events with M > 5½ are possible, it is likely that they would involve fault rupture extending downward into the carboniferous. This suggests the need to continue to improve the physics-based earthquake source and rate model. New data on the state of stress would, in particular, be extremely valuable, as would an improved understanding of the locations and source processes of the microearthquakes.

References

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Bourne, S.J., Oates, S.J., 2014. An activity rate model of induced seismicity within the Groningen Field. Technical Report. Nederlandse Aardolie Maatschappij. Assen, The Netherlands.

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Bourne, S.J., Oates, S.J., 2015b, An activity rate model of induced seismicity within the Groningen Field (Part 2), Nederlandse Aardolie Maatschappij. Assen, The Netherlands, July 2015.

Darendeli, M., 2001, Development of a new family of normalized modulus reduction and material damping curves. Ph.D. Thesis, Dept. of Civil Eng., University of Texas, Austin, TX.

Pickering, M., 2015, An estimate of the earthquake hypocenter locations in the Groningen Gas Field, Nederlandse Aardolie Maatschappij. Assen, The Netherlands, June 2015.

Hi [redacted],

I wanted to pass along to you an insight I had recently about the influence of the specific spectral model for the earthquake source on ground motion. I suspect that this comes too late for your review, but did want to mention it to you.

I was recently reviewing work one of our postdocs is doing on the spectra of induced earthquakes. Her work shows that, at least for one high quality data set, the 1980 spectral model of Boatwright fits the spectral shape of the data significantly better than Brune's 1970 model.

The equation for the Brune model is

$$u_1(f) = \frac{Mo}{1 + (\frac{f}{f_{c1}})^2}$$

and that for the Boatwright model is

$$u_1(f) = \frac{Mo}{\sqrt[2]{1 + (\frac{f}{f_{c1}})^4}}$$

Boatwright's model has a sharper corner than Brune's model and as a consequence radiates more energy near the corner for the same seismic moment and high-frequency acceleration asymptote. At the corner, the amplitude is 1.4 times that of the Brune model (actually sqrt(2)), and the total radiated energy is greater by a factor of about 3.25.

Earlier studies by Abercrombie found that both models fit data equally well, although the data was not of the same quality. So, at a minimum, the Boatwright model should be considered as a candidate for the source spectrum.

By considering only the Brune model, the GMPEs presented by Bommer et al., may be underestimating the ground motions. This could be handled by logic tree branches (epistemic uncertainty), but might also be resolved by careful analysis of the seismograms if the data is of sufficient quality. Perhaps this is something that we at Stanford could investigate if there was an interest.

Let me know if you have any questions about this or the report.

Cheers,

Bill

Here's the reference for Boatwright:

Boatwright, John. "A spectral theory for circular seismic sources; simple estimates of source dimension, dynamic stress drop, and radiated seismic energy." *Bulletin of the Seismological Society of America* 70, no. 1 (1980): 1-27.

Appendix I Review of Building Strength and Frality Workstream – Ron O. Hamburger

Review of Proposed Seismic Risk Study

Groningen Field
Groningen, Hollan
3 September 2015

SGH Project 157205

SIMPSON GUMPERTZ & HEGER
Engineering of Structures
and Building Enclosures

PREPARED FOR:

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CORP-GPS-433
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T A B L E O F C O N T E N T S

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ILLUSTRATIONS

3 September 2015

Mr. J. Ward Turner
Senior Technical Professional
ExxonMobil Development Company
PO Box 4876
CORP-GPS-433
Houston, TX

Project 157205 – Groningen Field Seismic Risk Study

Re: Report of Findings and Conclusions

Dear Ward:

We are pleased to transmit the attached report of our findings and conclusions following our review of documentation provided by you on the subject study.

We look forward to receiving your comments on this report.

Sincerely yours,

Ronald O. Hamburger, S.E.
Senior Principal
CA License No. 2951
I:\SF\Projects\2015\157205.00-GRON\WP\002ROHamburger-L-157205.00.jdi_Transmittal.docx

Encls.

ABSTRACT

Exxon/Mobil, Shell, and the Dutch Government are partners in operation of the Groningen field, the largest operating natural gas production field in Europe. Production initiated in 1962. In the mid-1980s, the field began to experience low level seismicity, associated with the production operations. This seismicity has continued to this day and has been observed to vary with the amount of production. Presently, production operations at the field have been reduced, at the insistence of the Dutch government, as a means of limiting the potential for a significant earthquake disaster in the region. The Joint Venture (NAM) is in the process of conducting a probabilistic seismic risk analysis (PRA) to quantify the potential for building collapse and life endangerment as a result of the induced seismicity. It is anticipated that a program of building retrofit will be undertaken to reduce the computed risk.

Consultants to NAM are presently building the risk model that will be used to perform the PRA. This includes quantification of the seismic hazards, the fragility of structures located in the affected area, and the exposure of the public to fatality, given building collapse. Fragility development includes an extensive program of analytical work supplemented by laboratory testing.

Exxon/Mobil retained Simpson Gumpertz & Heger Inc. to review selected documentation for the risk study and to provide an opinion as to whether the proposed approach was likely to provide meaningful estimates of the risk. We reviewed a series of PowerPoint presentations and reports provided by Exxon/Mobil, principally documenting analytical and laboratory work presently underway, or recently completed.

It appears the proposed PRA will employ state of art procedures developed and adopted by the industry for region-wide risk assessment projects. These procedures do inherently employ some conservatism which can overstate losses at low levels of ground motion intensity. Further, while fragility information presented for some building types appears to match well with past work by others, the fragilities for some building types, notably URM buildings with wood diaphragms, appears to indicate substantially higher probability of collapse than suggested by others for similar structural types.

Finally, the proposed limits for acceptable risk have a lower threshold for acceptable losses than have been adopted in the U.S.

REVIEW OF PROPOSED
SEISMIC RISK STUDY
GRONINGEN FIELD
GRONINGEN, HOLLAND

1. INTRODUCTION

1.1 BACKGROUND

Nederlandsse Aardolie Maatschapij BV (NAM), a joint venture of ExxonMobil and Shell are joint operators of the Groningen Gas Field, in the Netherlands. The partnership has been continuously producing natural gas from the field since 1962. Long term extraction of natural gas led to wide spread settlement and subsidence of the field and starting in 1986, frequent occurrence of earthquakes. Earthquakes to date have been small magnitude, the largest being M3.6. However seismic hazard studies based on the observed seismicity in the field, project the potential for larger magnitude events. Prior to large scale production at the field, the Groningen area has not been subject to appreciable earthquake activity. Consequently, building practices have not included earthquake-resistant construction and many of the buildings in the region are of unreinforced masonry, precast concrete and non-ductile concrete construction – types known to be vulnerable to earthquake damage. Consequently, NAM, under urging from the Dutch government has undertaken probabilistic risk assessments to quantify the risk of earthquake induced structural collapse and life loss in the region.

ExxonMobil retained Simpson Gumpertz & Heger (SGH) to perform review of selected data produced by NAM to document the study, and to provide comment on the validity of the proposed approach.

1.2 Objective

The objective our study is to provide ExxonMobil with an independent expert opinion as to whether the probabilistic risk assessment undertaken by NAM will provide a reasonable and meaningful characterization of the regional seismic risk in and around the Groningen field. Our study is principally limited to consideration of the development of collapse fragility functions for representative building archetypes.

1.3 Scope of Work

Our scope of study included the following:

- Review of selected documents provided by ExxonMobil that describe the probabilistic risk assessment being under taken by NAM.
- Review similar methodologies developed for similar purposes in the U.S.
- Prepare this report providing our independent opinion on the validity and limitations of the proposed probabilistic risk assessment.

2. DOCUMENT REVIEW

We reviewed the following documents provided by ExxonMobil:

1. Bourne, S., Oates, S., Bommer, J., Crowley, H., Pinto, R: PowerPoint Presentation: "Progress Towards a Probabilistic Seismic Risk Analysis for the Groningen Field – A Monte Carlo method for PSRA", dated 8-9 April 2015.

2. Hardie, S. PowerPoint Presentation: "Team Groningen Aardbevingen, Risk Metrics – Output from Probabilistic Risk Model", dated 9 April 2015.

3. Pinho, R and Crowley, H. PowerPoint Presentation: "v1 non-URM structural modelling".

4. Pinho, R; Grant, D., Magenes, G, Penna, A. and Rots, J, Powerpoint Presentation: "Modelling of URM structures", dated 9 April 2015.

5. Pinho, R, Report: "Numerical evaluation of the seismic response of the main typologies of non-masonry (non-URM) buildings that are found within the Groningen region," dated 12 May 2014.

6. Arup, Report: Groningen Earthquakes – Structural Upgrading, Modeling and Analysis Cross-Validation – Arup, Eucentre, TU Delft, Draft Version as Pre-Read Material for Scientific Advisory Committee: Fragility and Risk Work Group Meeting on 26th March 2015", dated 19 March 2015.

7. Crowley, H and Grant, D, Powerpoint Presentation: "v1 fragility functions & fatality ratios".

8. Report (unattributed): "Exposure Model v1 – Updated Typologies and Inference Rules", dated 17 March 2015.

9. Pinho, R and Crowley, H. PowerPoint Presentation: "Non-URM Modeling".

10. Grant, D., Powerpoint Presentation: "v1 URM structural modeling".

11. Crowley, H and Grant, D. PowerPoint Presentation: Update on v1 fragility functions and fatality ratios".

12. Pavia Risk Centre, "Protocol for Shaking Table Test on Full Scale Building (Eucentre) V-1".

13. Pavia Risk Centre, Full-scale test-house (v.4_8_15).

14. Report (unattributed) "Selection of Acceleration Time-Series for Shake Table Testing of Groningen Masonry Building at the Eucentre, Pavia," dated 1 August 2015.

2.1 Bourne, et al. April 2015 (Document 1)

This PowerPoint presentation provides an overview of probabilistic seismic risk analysis planned to quantify potential building damage and associated injuries/casualties resulting from gas production-induced seismicity in the Groningen field.

Key elements of the analysis include:

1. Seismicity model that projects the number, magnitude and location of earthquakes, as a function of production rate.

2. Ground Motion Prediction Equations (GMPEs) used to predict the probable intensity of ground shaking at varying points throughout the field.

3. Exposure model describing the geographic distribution of buildings conforming to different archetypes (construction characteristics and size).

4. Fragility functions that relate the probability a building will be damaged to a particular state, or more severe state, as a function of ground motion intensity.

5. Consequence (Injury) functions that indicate the probability that individual building occupants will be injured given that one or more building damage states are reached (or exceeded).

Monte Carlo analysis is proposed in which:

1. Many earthquakes (realizations) of differing magnitudes are postulated, consistent with the seismicity model.

2. For each earthquake realization, the intensity of shaking is computed on a 3 km by 3 km grid across the field, using the GMPEs. We presume that intensity is computed as a random variable in this process.

3. At each grid point, the exposure model is used to determine the number of buildings of different types present.

4. For each realization, using the computed shaking intensities, the damage state sustained by each building is computed using the fragility functions. We presume that damage state is computed as a random variable.

5. For each building, the number of injuries is computed using the injury functions. Injuries are a random variable.

6. By summing the number of damaged buildings and injuries for each realization, and also over all realizations the distribution of and mean number of losses is computed.

By varying assumptions as to production rate, and also as to building fragility, it is possible to explore the effects on damage and casualty risk of varying the production rate and/or seismically retrofitting buildings in the region.

2.2 Hardie PowerPoint (Document 2)

This presentation discusses acceptable risk to life safety. The presentation notes that the probability that an individual will experience fatality due to accidental causes in the Netherlands today is approximately 1×10^{-4} per year. Presumably this includes accidents of all kinds including aircraft, automotive, industrial, etc. The presentation indicates the acceptable risk to life safety expressed in the draft Dutch Building Code (NEN-NPR-9998) is 1 x 10^{-5} per year. The presentation suggests that as a temporary measure, structural upgrade to achieve a life safety risk to individuals of 1 x10^{-4} is appropriate.

2.3 Pinho and Crowley (Document 3)

This presentation documents modeling of building archetypes, other than unreinforced masonry archetypes, undertaken to establish structural fragility functions. Summary analysis data, in the form of pushover curves are presented for a series of building archetypes including:

- Precast Terraced Houses (REST-RC-B). These are two-story, residential buildings with structural systems comprising precast reinforced concrete walls supporting reinforced concrete slabs. Roofs are pitched construction. The buildings are rectangular, comprising a single row of two-story residential units. Variations of this archetype included a four-story and seven-story archetypes, and buildings with and without foundation beams to tie walls together; and two different values of friction coefficient between the walls and foundations. Ultimate displacement capacities of 0.01m is predicted for archetypes with foundation beams; dominated by shear failure of walls. Ultimate displacement capacities of 0.05m are predicted for archetypes without foundation beams, in which failure is dominated by sliding of the walls relative to the foundations.

- Cast-in-place Terraced houses (REST-RC-A). These are two-story residential buildings, constructed using tunnel-formed reinforce concrete. Lateral resistance is provided by load bearing shear walls in the transverse direction and frame action of the slabs and walls in the longitudinal direction. Estimated ultimate displacement capacity is approximately 0.14 m limited by shear failure of acting as columns of portal frames.

- Cast-in-place reinforced concrete apartment buildings, greater than four stories in height (RESA-RC-A-G4S). Ultimate displacement capacities varying between 0.02 m and 0.035 m are presented based on building configuration. The limiting behavior mode is not indicated.

- Cast-in-place reinforced concrete apartment buildings, less than four stories tall (RESA-RC-A-L4S). These wall type buildings are limited by wall shear behavior and have analytically computed ultimate displacement capacities on the order of 0.01 m.

- Detached, wood-frame single family residence structures (RESD-W-A). These two-story wood frame structures with gypsum sheathing have analytically predicted displacement capacities of 0.25 m. The limiting behavioral mode is not indicated.

- Single-story, high bay steel-frame agricultural buildings (AGRI-INDU-COML-S-A and B) consisting of gabled moment-resisting steel frames in the transverse direction and light steel braced frames in the longitudinal direction. Ultimate displacement capacities of up to 1 m are analytically predicted. Plastic hinging of columns and buckling of braces are the limiting behavioral mode.

- Single-story, high bay wood-frame agricultural buildings with unreinforced masonry veneer (AGRI-INDU-COML-W-A). Ultimate predicted displacement capacities for these buildings approximate 0.04 m. The limiting failure mode is not indicated.

- Single-story, high bay, glulam portal frame buildings (AGRI-INDU-COML-W-B1). The structural system consists of wood portal frames in the transverse direction and steel bracing in the longitudinal direction. Ultimate behavior is limited by failure of semi-rigid connections at column bases and buckling of braces. Ultimate displacement capacities of 0.3 m in the transverse and 0.7 m in the longitudinal directions are predicted.

- Single-story, high bay, glulam portal frames with infill unreinforced masonry walls (AGRI-INDU-COML-W-B2). Failure is triggered by column base connections in transverse response and infill masonry wall failure in the longitudinal direction. Ultimate displacement of 0.1 m is projected in the longitudinal direction and 0.3 m in the transverse direction.

- Single-story, industrial/agricultural buildings constructed with precast concrete frames supporting light steel roofs (AGRI-INDU-COML-RC-B1). Ultimate failure modes identified include unseating of roof beams and column rotational modes. Median ultimate displacement capacities, based on incremental dynamic analysis of 0.3 m are projected.

- Single-story, industrial/agricultural, cast-in-place concrete buildings. Lateral resistance is provided by portal frame behavior of columns, slabs and beams. Median ultimate displacement capacities, based on rotation at column bases, of 0.6 m is projected using incremental dynamic analysis.

- Precast reinforced concrete wall commercial buildings less than or equal to four stories in height (COMO-RC-B-L4S). Limited failure modes are not indicated. Ultimate displacement capacities presented range from 0.03 m to 0.045 m.

- Steel moment frame commercial buildings less than or equal to four stories in height (COMO-S-B-L4S). Ultimate behavior of these buildings is limited by exceeding the rotational capacity of columns at their bases. Ultimate displacement capacity ranging from 0.3 m to 0.4 m is presented.

2.4 Pinho et al. (Document 4)

This presentation describes analytical modeling conducted of unreinforced masonry (URM) structures noting that the types of URM structures present in the Groningen field are unique and atypical of URM buildings for which existing fragility data exists. The presentation also describes analytical modeling of laboratory tests of representative calcium silicate masonry wall panels, used to validate, evaluate and benchmark analytical models used to develop fragility functions for URM buildings. Capacity curves, indicating ultimate spectral displacement capacity for a series of structures are presented including:

- Terraced clay brick residential structure with timber floors (REST-URM-A-4U). Construction of this archetype appears similar to clay brick URM buildings worldwide. Spectral displacement capacities of 0.02 m are projected.

- Detached house with solid walls and timber floors (single story) (RESD-URM-A) for which a spectral displacement capacity of 0.14 m is projected.

- Residential solid clay brick walls with timber floors (RESA-URM-A-L4S), four stories or less. Construction of this archetype appears similar to clay brick URM buildings worldwide. An ultimate spectral displacement capacity of 0.025 m is projected.

- Residential building with calcium-silicate walls and concrete floors and roof (RESA-URM-B-L4S). A spectral displacement capacity of 0.02 m is projected.

- Industrial, single story building. No details of construction are presented. Projected ultimate spectral displacement capacity is indicated as 0.012 m.

2.5 Pinho Numerical Evaluation (Document 5)

This report presents the results of a series of structural analyses of building systems and entire buildings that have been subjected to laboratory testing, and used to benchmark and calibrate analytical models used to form structural fragility functions. Structural model types evaluated include:

- Precast concrete frames. Figure 15 plots base-shear vs roof displacement hysteresis for the analysis and a benchmark test. Agreement is good, although hysteretic data indicates that response is essentially elastic.

- Cross Laminated Timber Panels. Figure 20 plots predicted hysteresis from the analysis against laboratory data showing good match. However, the hysteretic data remains within the strain hardening range of response, with no strength degradation evident.

- Masonry infill panels with reinforced concrete frame. The experiment consisted of a shake table test of a model building. Figure 27 presents comparison of roof displacement time histories for the experiment and analysis. The dynamic phasing of predicted response matches the experiment well. Amplitudes of roof displacement closely match the experiment for some cycles and not for others. Data is not presented to indicate the extent of inelastic response occurring, however, residual displacement is small indicating that inelasticity is limited.

- Cast-in-place reinforced concrete frame. The experiment consisted of a shake table test of a model four-story frame. Figures 35, 36 and 43 compare the roof displacement time history for the experiment and analysis for two different ground motion inputs. The analysis and experiment compare closely. No data is presented to indicate the extent of nonlinearity that occurred in the experiments.

- Reinforced concrete wall structure. The experiment consisted of a shake table test of a seven-story concrete shear wall building. Figure 50 presents roof displacement time histories for the experiment and analytical prediction for EQ4, one of several ground motions used in the experiment. Analytical prediction of behavior is good through five cycles of strong response, after which time, phasing and amplitude of the predicted and recorded response diverge. There is no indication as to the amount of damage that occurred during the test.

- Steel moment-resisting frame. The experiment consists of a three-story steel moment frame structure on a shake table. Figures 58 through 60 present hysteretic plots on a story basis for the analysis and test. The analysis fails to predict hysteretic pinching observed in the test in the 1st and 2nd stories. The analysis over-predicts displacement in the top story. No degradation, other than hysteretic pinching is evident in the figures. The structure appears to be responding within the strain hardening as opposed to strength degrading range.

- Steel Moment Resisting Connections. Analytical benchmarks of several tests quasi-static, cyclic tests of moment-resisting beam-column connections are presented. Hysteretic comparisons are presented in Figures 79 through 81. Matching of hysteretic shape and amplitude is good including, in the case of Figure 80, hysteretic response is within the strength degrading range.

2.6 Modeling and Analysis Cross Validation (Document 6)

This report documents the results of a study under which four different approaches to modeling the seismic response of unreinforced masonry walls and structures was undertaken. In the study, analyses, using the different approaches, were undertaken to predict the behavior of actual tested components and structures. Components tested included:

- Single wall panel of solid clay brick masonry subjected to in-plane shearing.
- Single wall panel of single leaf calcium silicate masonry subjected to in-plane shearing.
- Single wall panel of one-way spanning, single leaf unreinforced clay brick masonry subjected to out-of-plane loading.
- Single wall panel of two-way spanning, singe leaf clay brick masonry subjected to quasi-static out-of-plane loading.
- A full-scale, two-story building subjected to quasi-static loading.
- A full-scale calcium silicate brick building.

The report notes that agreement between the team's predictions of behavior was low and that additional blind testing is necessary to demonstrate robust and appropriate modeling technique.

2.7 Crowley, et al. V1 Fragility Functions (Document 7)

This PowerPoint presentation summarizes the development of fragility functions for ninety-four building archetypes. These curves will use spectral acceleration at the building's effective period to characterize ground motion intensity. This presentation also summarizes the considerations associated with generating probable fatalities given that a building is predicted to collapse. This involves estimating the fraction of time a person is within an area exposed to building debris, and the percent of a building's volume that is subject to debris accumulation given that collapse occurs.

3. DISCUSSION

3.1 Risk Modeling Methodology

The overall risk modeling methodology described in the documents follows well established procedures that have been implemented a number of times in the United States and abroad, by government agencies as well as private enterprises, primarily in the insurance industry. The basic procedure is best summarized in scholarly works produced by the Pacific Earthquake Engineering Research Center including Moehle (2009).[1] As discussed by Moehle, the probability of incurring a loss *P(loss)* as a result of earthquake induced damage for a single building is given by:

$$P(loss) = \iiint \{P(L|DS)\}\{P(DS|EDP)\}\{P(EDP|Ha(z))\}dz \tag{1}$$

where, $H_a(z)$ the hazard curve, portraying the intensity of motion at probability z; EDP is the value of an engineering demand parameter indicative of performance, such as drift; DS is the occurrence of a damage state, such as building collapse, $P(DS|EDP)$ is the fragility function; L is the probable loss given the occurrence of damage state DS; and $P(L|DS)$ is the loss function. This same approach, developed to portray the loss associated with an individual building's performance can be used to portray the probable loss to an entire community or region by summing equation (1) over all buildings in the region:

$$P(loss) = \sum \iiint \{P(L|DS)\}\{P(DS|EDP)\}\{P(EDP|Ha(z))\}dz \tag{2}$$

While closed form solutions to equations (1) and (2) are possible, if each of the random variables can be characterized in a suitable functional form, most commonly these equations are solved through Monte Carlo procedures as is proposed.

The earliest implementation of risk models such as those given by Equations (1) and (2) were for the insurance industry, who desired to quantify their potential losses due to earthquakes and other hazards. Steinbrugge[2] was among the first to publish information on this approach including rudimentary fragility functions that expressed probable repair cost as a function of earthquake intensity, where earthquake intensity was characterized by Modified Mercali

[1] Moehle, J.P. and Deirelein, G.G., (2003) A Framework Methodology for Performance-based Earthquake Engineering, *13th World Conference on Earthquake Engineering*, **Proceedings**, Paper 679

[2] Steinbrugge, KV, Earthquakes, Volcanoes and Tsunamis: An Anatomy of Hazards, Skandia America Group, 1982.

Intensity (MMI). In 1985, ATC published its landmark ATC-13[3] report which provided suggested fragility functions for 78 classes of California buildings developed on the basis of expert opinion of a panel of experts. Like the Steinbrugge approach, these fragility functions directly related repair cost, expressed as a percentage of building replacement cost, to MMI.

In the early 1990s, the U.S. Federal Emergency Management Agency entered into contract with the National Institute of Building Sciences to develop a national seismic hazard and loss estimation model, known as HAZUS. FEMA developed this model to enable communities in seismically active regions of the U.S. to understand the risk of earthquake induced losses with the hope that such knowledge would assist in the adoption of effective earthquake risk mitigation measures. The HAZUS earthquake model includes a detailed methodology, described in technical manual, and accompanying software that is available for free from the U.S. Government. The HAZUS methodology embodies many of the same procedures described in the documentation reviewed, including:

- Use of the capacity spectrum method, in which pushover curves and response spectra, plotted in acceleration vs. displacement domain are used to characterize structural fragility.
- Use of a series of model building archetypes to represent an entire population of buildings.
- Use of gridded seismic intensity calculations to compute the losses for a scenario earthquake.
- Use of Monte Carlo procedures to compute earthquake losses.

Enhancements to the HAZUS methodology that Nam has undertaken in their loss modeling appear to include:

- Development of region-specific fragility and consequence functions.
- Use of magnitude-dependent fragility functions.
- Development of hazard functions representing production-dependent, induced seismicity.

It is important to note that before publication of HAZUS for use, the development team undertook a significant benchmarking effort to verify that the results produced by HAZUS were compatible with actual experience in recent earthquakes. Notably, a benchmark study against

[3] Applied Technology Council. *Earthquake Damage Evaluation Data for California ATC-13*, Federal Emergency Management Agency, 1985.

recorded losses in the 1994 Northridge earthquake was undertaken, ultimately resulting in adjustment of fragility and loss functions in order to calibrate the model to the Northridge data.

Contemporary with the development of HAZUS, several private companies including EQECAT, and Risk Management Solutions independently developed earthquake loss models, primarily focused at assisting the insurance industry to undertake loss estimates for their portfolios of insured properties. Originally these loss models used fragility and loss functions based on the ATC-13 publication, with some proprietary modification. Later these models adopted improved fragility functions, incorporating some of the HAZUS methodology procedures. However, since these models are proprietary, the fragility and consequence functions are not available for use as benchmark for this project.

3.2 Building Archetypes

The NAM study includes a significant component associated with development of fragility and consequence functions appropriate to the building inventory in the Groningen field on the basis that construction present in this region is unique and that earthquake fragility functions developed for other locations, e.g. the United States are not applicable. Based on the information presented it appears that several of the selected archetypes are sufficiently similar to construction in other regions that the fragility functions used for buildings in other regions could be directly applicable, or as a minimum, should be used to inform opinions as to whether the Groningen fragility functions are appropriate. These include:

- Cast-in-place reinforced concrete apartment buildings (RESA-RC-A-L4s), (RESA-RC-A-G4S).
- Detached wood-frame single family residence structures (RSD W-A).
- Single story, high bay steel-frame agricultural/industrial buildings (AGRI-INDU-COML-S-A and B).
- Single story, cast-in-place concrete industrial buildings ((AGRI-INDU-COML-RC-B).
- Steel moment frame commercial buildings less than four stories in height (COMO-SB-L4S).
- Residential solid clay brick walls with timber floors (RESA-URM-A-L4S).

3.3 Fragility Development

There are presently four commonly accepted methods of determining structural collapse fragilities for buildings. These include the HAZUS method; incremental dynamic analysis; earthquake experience data; and, engineering judgment;.

3.3.1 HAZUS Method

In the HAZUS methodology nonlinear static (pushover) analysis is used to evaluate the force-deformation behavior of a structure when subjected to a monotonically increased static loading pattern. Under the technique, the structure is analytically "pushed" with the stiffness matrix adjusted to represent the onset of yielding, buckling and other structural damage. Collapse is generally judged to occur when the analytical model becomes unstable, e.g. when a critical load bearing column fails and load is unable to redistribute; or when the deformation and strength demands predicted by the analysis on one or more "critical" elements exceed limiting values at which loss of load reliable load carrying capacity is expected. As the progressively increased analyses are performed, a plot of applied shear force vs. displacement of a reference node is plotted. Using modal mass and shape factors, the pushover curve is converted to Spectral Acceleration v. Spectral Displacement (ADRS) coordinates, in which form it is known as a capacity curve. The capacity curve is plotted on top of a demand spectrum, which is adjusted to represent progressively increased effective damping with increasing displacement and nonlinear behavior. The point at which the capacity curve and appropriately damped response spectrum intersect defines the "performance point" at which ground motion represented by the spectrum would push the structure. The amplitude of the spectrum that produces a performance point at the indicated "point of collapse" on the capacity curve is indicative of the collapse capacity for the structure. Uncertainties are then aggregated considering issues of record to record variability, modeling uncertainty, etc. The resulting spectral amplitude and uncertainty completely define the collapse fragility.

Issues with the use of the HAZSUS procedure to establish structural fragility include the following:

1. The validity of the capacity curve depends greatly on the extent the analytical model represents reality.
2. The performance point solution for earthquake demand is at best approximate and subject to significant uncertainty, particularly for structures with periods in the constant response acceleration (short period) domain. Work by Bertero, Miranda, and many others suggests that ductility demands on short period structures become very large at

relatively modest strength ratios. This leads to conclusions, when this approach is used that collapse is very likely for such short period structures under even modestly strong motions. However, past experience in real earthquakes does not align with these analytical predictions. Many engineers have postulated possible reasons for this lack of agreement between observed performance and analytically predicted performance, sometimes known as the "short period paradox" including soil nonlinearity, soil structure interaction, structural overstrength that is not fully accounted for in the analytical models, and other reasons. The Applied Technology Council is presently engaged in a project that will attempt to resolve this short period paradox. Meanwhile, it is undisputed in the industry that analytical predictions of collapse for such short period structures tend to greatly over-estimate collapse potential.

3. The use of nonlinear deformation limits to indicate onset of collapse, as is typically done in the HAZUS approach is conservative and tends to overestimate true structural fragility. These deformation limits are typically chosen in the laboratory based on 1) observation of the specimen's condition during testing; and 2) arbitrary limits on strength loss, such as 80%, 65% or similar percentage of peak strength. The reasons for this are that laboratories are unwilling to test structures to true failure, because it is destructive of laboratory equipment and dangerous, and also, because it is judged that once significant strength degradation occurs, the structure will become unstable and collapse. This judgement often tends to be very conservative. Figure 1 below is illustrative of this. This photo shows one of several shear walls in the Edificio Toledo building in Vina del Mar, Chile, following the 2011 earthquake. This subduction zone earthquake produced more than 2 minutes of strong shaking. The wall shown in this photo completely crushed the concrete for a height of approximately 18 in. at the base of the wall, and also suffered buckled and fractured reinforcing as did most other walls in the building, yet the building remained quite stable and did not collapse. Standard analytical procedures used in developing HAZUS type fragility curves would have predicted collapse at a fraction of the spectral demands actually experienced by this building. Many examples of similar behavior can be found.

Figure 1: Failed shear wall in Edificio Toledo, Vina del Mar, Chile, 2011

3.3.2 IDA Technique

IDA is a powerful analytical tool for determining collapse fragility. It consists of developing a nonlinear analytical structural model and subjecting the model to nonlinear dynamic analysis using a suite of appropriate ground motions. Each ground motion is incremented in amplitude, until the analysis predicts collapse, either directly; by predicting drift, strength or deformation demands that would result in collapse; or, by resulting in analytical instability. The ground motion intensity that results in 50% of the records producing collapse is taken as the median intensity for collapse. The fragility is then constructed by associating a standard deviation, in log space, to the median capacity. FEMA 350[4] and FEMA P695[5] both describe the use of IDA to develop structural fragilities.

The use of IDA to determine collapse fragility is subject to many of the same limitations as the HAZUS approach. The analytical prediction is only as good at the ability of the model and its

[4] SAC Joint Venture, *Guidelines for Design of New Moment Resisting Steel Frame Buildings, FEMA 350*, Federal Emergency Management Agency, Washington, D.C., August, 2000
[5] Applied Technology Council, *Quantification of Building Seismic Performance Factors, FEMA P695*, Federal Emergency Management Agency, Washington, D.C., June, 2009

hysteretic elements to predict behavior. It is important to note that many nonlinear models provide reasonable response predictions that are reasonably similar to that observed in the laboratory at moderate levels of inelasticity but become less valid at extreme nonlinear response. This is significant because collapse is a form of extreme nonlinear response. Modeling techniques that work well at modest levels of inelastic behavior do not necessarily predict collapse well.

In addition to the modeling limitations described above, in many IDA analyses, collapse is taken as the onset of limiting deformation demands on “critical” elements, sometimes called non-simulated or inferred collapse. For the same reasons discuss in the previous section these limiting deformation demands tend to be overly conservative.

A final note on the use of IDAs to predict collapse fragility is that even with perfect hysteretic elements, analytical structural models will typically include much of the stiffness and damping inherent in real building structures. This additional stiffness and damping is a result of structural elements that are not considered to be part of the lateral force-resisting system, but which nevertheless do contribute strength and stiffness, and also result from nonstructural walls, cladding, stairs and other elements. Comparisons of analytical prediction of structural period with periods derived using signal identification techniques on strong motion recordings has demonstrated repeatedly that analytical predictions can over-estimate structural period by 40% or more depending on the structural type. Given that actual inelastic displacement demand varies approximately with the square of structural period, it can be seen that analytical models can substantially over predict displacement demands, and resulting damage or onset of collapse.

3.3.3 Engineering Judgment

Engineering judgment was the original method of establishing collapse fragilities for structures and was used in establishment of fragilities by Steinbrugge and ATC-13. It is typically informed by observation of real performance in earthquakes but is hampered by the fact that recorded ground motion records are rarely present at the sites of collapsed buildings and therefore, engineers must guesstimate the ground motion experienced by buildings included in the observational data base upon which their judgment is based. While of obviously questionable accuracy, engineering judgment can be used as a sanity check on fragilities that have been derived by other means.

The well-known MMI and other intensity scales are well known examples of such means of determining structural fragility. In the case of MMI, an intensity of VII is defined as resulting in considerable damage in poorly built or badly designed structures; while MMI VIII is defined as producing partial collapse in ordinary structures and great damage in poorly built structures. Generally, when the MMI scale was constructed, building construction consisted primarily of wood frame and unreinforced masonry and therefore, is at least partially applicable to many of the structures in the Groningen field. Wald, et al.[6] provide approximate relationships to relate MMI to peak ground acceleration and velocity. Such relationships can be used to provide sanity checks to fragilities derived by other means.

3.3.4 Earthquake Experience Data

Perhaps the most direct way of establishing collapse fragilities is through observation of damage sustained by real buildings subjected to known shaking intensities. Given sufficient observation points is possible to directly apply earthquake observation data to construct fragilities by taking both a mean and standard deviation from the observations. Problems with this approach include:

1. It is rarely possible to find a statistically significant sample of similar buildings that have experienced known intensities of motion and collapse, permitting a valid fragility estimate.

2. When a building is observed to have collapsed in an earthquake, at a given intensity of motion, it is known that the motion was sufficient to induce collapse, but it is not known if less intense motion would also have produced collapse. Thus fragilities conducted in this way will likely underestimate the actual collapse fragility.

Given the above, it is actually preferable to use lack of occurrence of a given damage state, e.g. collapse, in a population of buildings subjected to known ground motion to establish a lower bound value on the collapse capacity of these buildings, rather than a median value, and then impute an uncertainty to this lower bound. This approach is commonly used in the nuclear industry, wherein a so-called HCLPF (high confidence, low probability of failure) capacity is determined based on observations of lack of damage in specimens subject to known loading intensities. In the nuclear industry, HCLPF is typically taken as a loading value that represents 95% confidence of less than a 5% chance of failure, or when epistemic and aleatory uncertainties are considered jointly, approximately a 2% probability of collapse.

[6] Wald, DJ, Quitoriarno, V, Heaton, TH, and Kanamori, H, "Relationships between Peak Ground Acceleration, Peak Ground Velocity, and Modified Mercalli Intensity in California", *Earthquake Spectra*, Volume 15 No. 3, August 1999, EERI, Oakland, CA

The use of a HCLPF approach is beneficial in that it overcomes some issues associated with the selection of lognormal distributions to represent structural fragilities. Lognormal distributions are used to represent fragility because 1) this functional type will never predict a non-null probability of damage at null ground motion intensity, 2) it is skewed, which has been observed to represent the true distribution of damage in buildings in past earthquakes, 3) it is mathematically convenient for use in closed form solutions to Equation 1 and similar expressions. Regardless, the true functional shape of fragility distributions is likely not lognormal. Many engineers have observed when performing loss studies like that proposed for Groningen that the lower tails of the lognormal fragilities seem to produce non-credible predictions of failure probability at low ground motion intensity. This is particularly important for studies of this type as relatively low levels of ground motion can dominate the predicted losses, particularly when thousands of buildings are involved, i.e., a small fraction of a very large number of buildings predicted to experience collapse is still a large number.

3.3.5 NAM Approach

Based on the documents reviewed, the approach adopted by NAM's consultants appears to follow that used in HAZUS, tempered by observational data, and IDA study. Since there have been no collapses due to induced seismicity in the Groningen field, observational data is scant, and at best, can be used to establish a HCLPF for collapse for the various structures in the exposed data base.

3.4 Acceptable Risk

The acceptable risk of collapse and life loss due to earthquakes and other hazards is a societal rather than engineering issue. What is acceptable to one society may not necessarily be acceptable to another. Some building codes have quantified acceptable risk due to earthquake shaking. Documents reviewed indicate the draft Dutch Building Code has identified 10^{-5} per year as the acceptable life safety risk for new buildings and notes a recommendation of 10^{-4} per year, as a temporary measure, in the Groningen field.

It may be instructive to compare these against similar risk adopted in the U.S. and other countries. In the United States, seismic hazard maps adopted by the ASCE 7-10[7] standard are based on a notional 2×10^{-4} per year collapse rate for ordinary occupancy buildings including most residential and commercial structures. Given that individuals are located in one structure

[7] American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures, ASCE 7-10*, ASCE, Reston, VA, 2010

or another, throughout most of the day, a uniform collapse risk at this level suggests a risk to individual lives of the same order.

Two things are worthy of note with regard to the acceptable collapse risk in the U.S. The first is that in regions with major active faults, such as San Francisco, Los Angeles, Salt Lake City the maps inherently accept a somewhat higher, but unquantified collapse risk by adopting deterministic caps that limit the probabilistically defined ground motion. Also, as a matter of public policy in the United States, municipalities are willing to accept somewhat higher risk of collapse in existing building than in new buildings. Historically, U.S. cities with significant seismic hazards have accepted retrofit of existing buildings to 75% of the strength required of new buildings. This implies that the spectral acceleration resulting in defined collapse probability of such structures would be approximately 75% of that for new structures, and that the resulting risk of collapse and life endangerment would be proportionately higher. This same philosophy is carried forward in the national standard[8] for seismic rehabilitation of buildings, which adopts design ground motions for existing buildings that are substantially reduced from those required for design of new buildings.

[8] American Society of Civil Engineers, *Seismic Rehabilitation of Buildings, ASCE 41-13*, ASCE, Reston, VA, 2013

4. CONCLUSIONS

The approach taken by NAM's consultants to characterize the risk of earthquake-induced structural collapse and fatalities due to induced seismicity in the Groningen region adopts the best current practices for such studies currently undertaken worldwide by both government and private parties. However, this state of practice is not perfect and has been known in past studies to over-predict potential losses, particularly for low level events. In specific we offer the following comments:

1. Benchmarking of analytical work against laboratory specimens undertaken by NAMS's consultants is admirable and appropriate. However, for reasons previously discussed, this benchmarking appears to be occurring for relatively low levels of nonlinear response. Collapse is a result of extreme nonlinear response. Benchmarking of the type undertaken by NAM's consultants does not necessarily demonstrate that analytical predictions of collapse are correct in that it is not clear if the modeling approaches will track well at extreme response.

2. As noted in some of the benchmarking studies for masonry buildings, the several analytical approaches undertaken did not agree well with each other. This is indicative of the significant uncertainty associated with predicting response for complex, highly nonlinear systems.

3. Although the proposed study conforms to present best practices, it inherently incorporates a number of conservative biases common to all such studies. The result of these biases is that study will tend to over-predict the true risk of building collapse and life endangerment. Conservative biases we identified include:

    a. For structures with fundamental periods less than about half second or so, nonlinear dynamic analyses, and static procedures calibrated to dynamic analyses, including so-called R-μ-T relationships developed by Miranda and others, predict very large displacement ductility demands at relatively modest values of the inelastic demand ratio, R. As a result, nonlinear analyses inherently predict collapse of such structures at ground motions modestly larger than those that load these structures to their elastic limits. Observation of real structures in earthquakes suggests this behavior is not correct. The profession generally recognizes this and ascribes this discrepancy between analytical predictions and observed behavior to soil-structure interaction, modeling simplifications and other effects.
    b. Analytical prediction of collapse is very difficult. Often, analysts will use somewhat arbitrary indicators of structural failure, such as reduction in strength to a defined fraction of peak strength to signal collapse. These indicators are typically conservative and prematurely predict true structural failure.
    c. Analytical models often neglect many structural and nonstructural elements that add substantial stiffness and strength to buildings. This results in under-prediction of actual stiffness, over-estimation of period, and consequently, over-estimation of displacement demands induced by earthquake shaking.
    d. Fragility curves are assumed to have lognormal distribution. While commonly for this purpose, many believe that the lower tails of lognormal distributions predict small, but significant probability of failure at demand levels that would

not credibly cause structural failure. When applied to large portfolios of buildings this inevitably produces large potential losses at levels of ground motion less than that at which damage has historically been observed.

4. The capacities predicted for some building archetypes including single-story, high bay industrial buildings and detached single family residences appear credible. Capacities predicted for other archetypes including some of the URM and concrete archetypes appear to be unreasonably low particularly when compared against fragilities that have typically been developed by others for similar building archetypes. In particular for the URM archetypes of wood diaphragms, the indicated capacities appear very low. Studies of similar buildings in the U.S. has indicated that seismic response is typically dominated by response of the flexible wood diaphragms which exhibit displacement capacities considerably in excess of those indicated in the URM fragility reports reviewed. Spectral demand will more closely approximate the diaphragm deflection of such buildings than the wall deflection. It does not appear this has been accounted for in the studies undertaken to date.

5. It is essential to benchmark the fragilities ultimately derived for this study against engineering judgment and to examine predicted HCLPF points from the fragilities against credible values based on observational data.

6. The life safety goals suggested as a basis for retrofit would seem to be aggressive compared with standards for both new and existing buildings deemed acceptable in seismically active regions of the U.S.

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