European Commission

Horizon 2020
European Union funding
for Research & Innovation

Proposal # 700748

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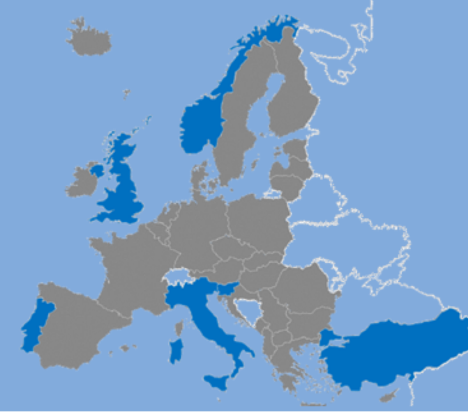

Consortium Meeting Pavia – October 2017

ASSESSMENT AND MITIGATION OF LIQUEFACTION POTENTIAL ACROSS EUROPE

A holistic approach to protect structures / infrastructures for improved resilience to earthquake-induced liquefaction disasters

WP3 - Structural Liquefaction Resilience & Vulnerability Assessment Methodologies

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CONSTRUCT

WP3 - Structural Liquefaction Resilience & Vulnerability Assessment Methodologies – developed:

1. An efficient numerical procedure for the simulation of liquefaction-induced damage of buildings.

   Macro-mechanism approach reduces analysis time to 2 minutes compared to 48+ hours for effective stress analysis

2. An efficient probabilistic framework for liquefaction vulnerability analysis of buildings.

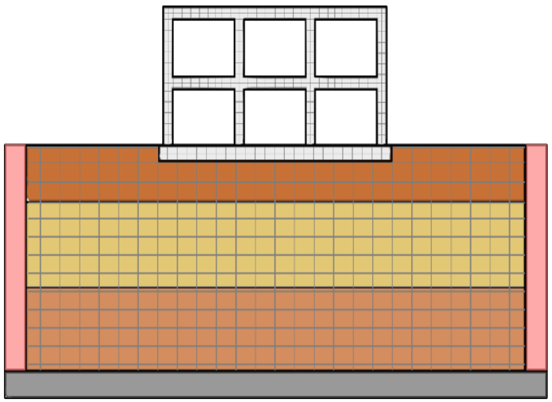
   Combination of building and soil profile classes with defined criteria allows an intuitive physics-based approach to assess vulnerability

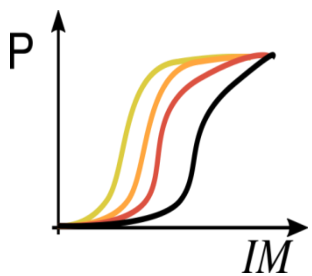
3. A general framework procedure for users and owners of buildings to assess subsoil properties and evaluate vulnerability.

   Vulnerability analysis framework works for regional and building specific studies, its modular design means additional accuracy or multiple approaches can be considered for each step

Simulation of options

Full model approach

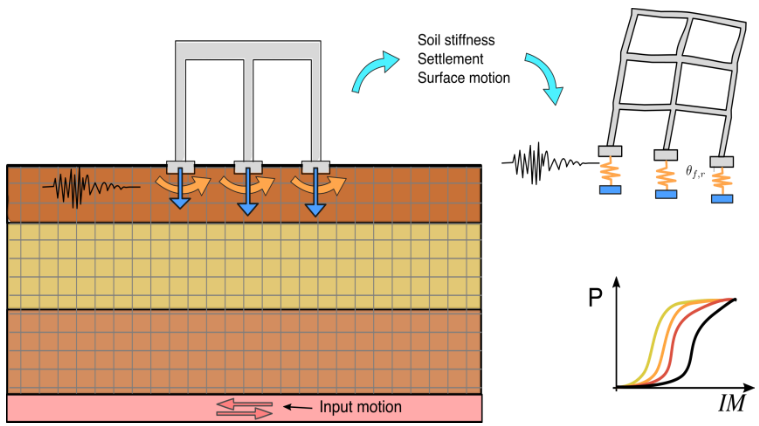




- Interactions implicitly dealt with
- Performance obtained from a single model

- Not efficient
- Limited structural modelling options
- Difficult to evaluate uncertainties

Macro-mechanism approach

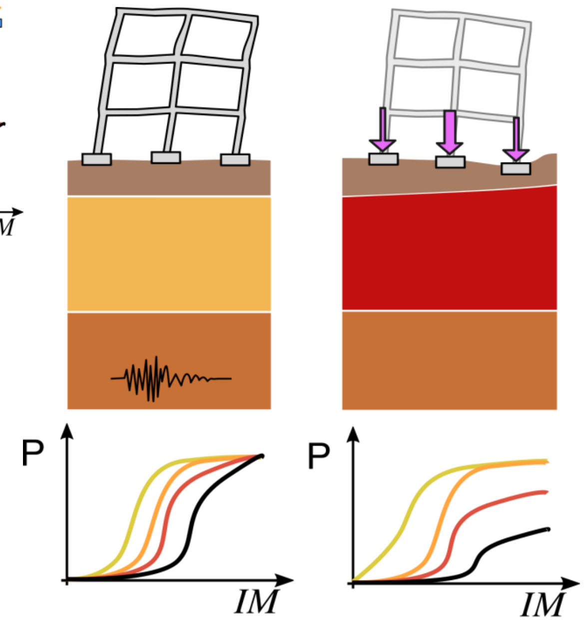


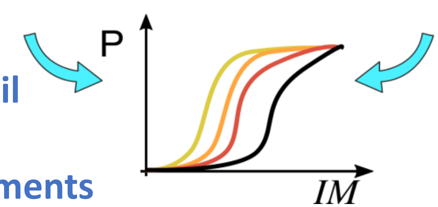
- Modular so can include multiple methods
- Uncertainties can be evaluated at each step

- Difficult to deal with nonlinearities appropriately
- Some aspects had not been quantified for immediate use

Separate hazards approach

- Fast to apply
- Can use existing shaking damage fragility curves

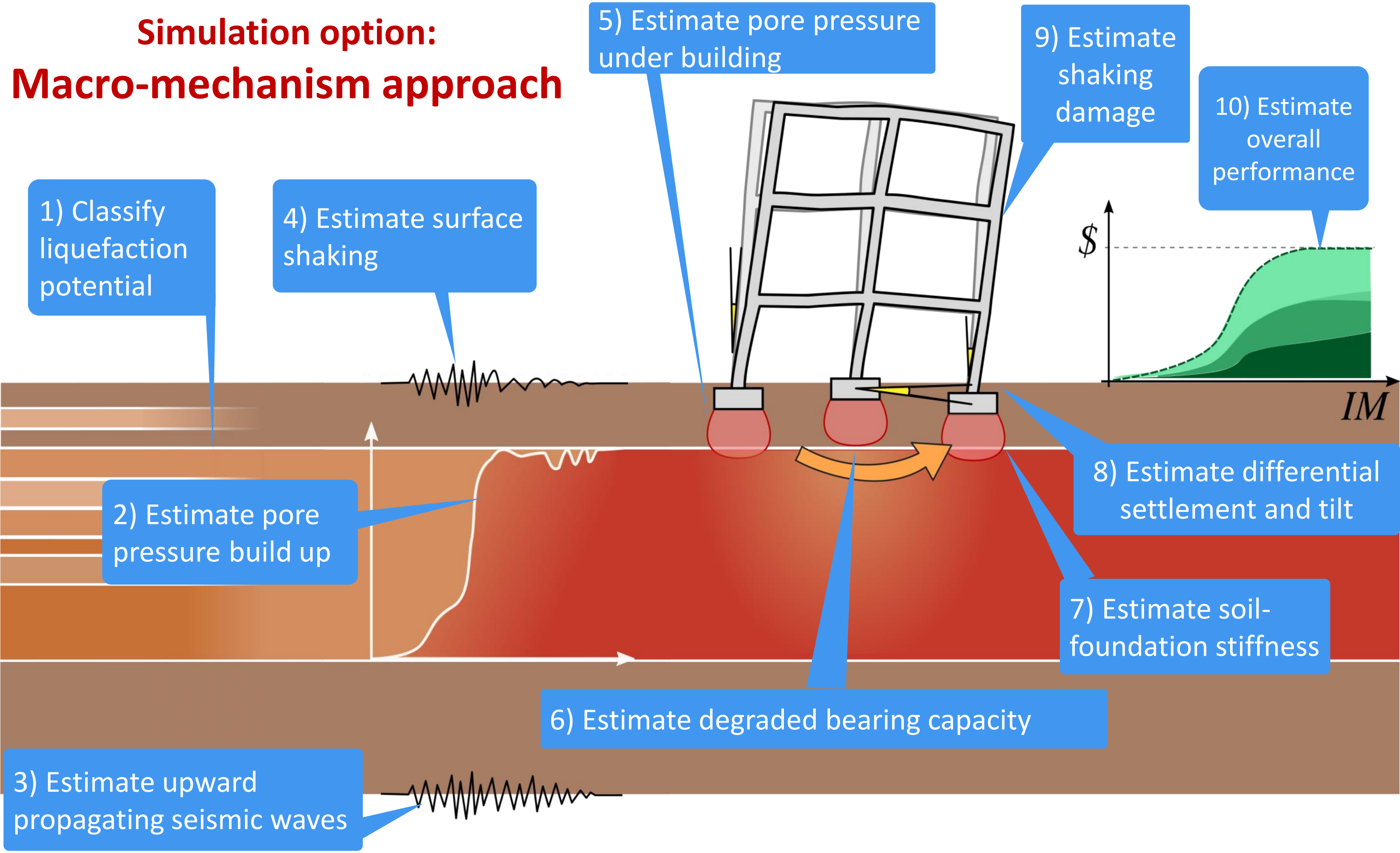




- No interaction between soil and *structure*
- Difficult to evaluate settlements
- Requires multiple intensity measures

Simulation option:

Macro-mechanism approach

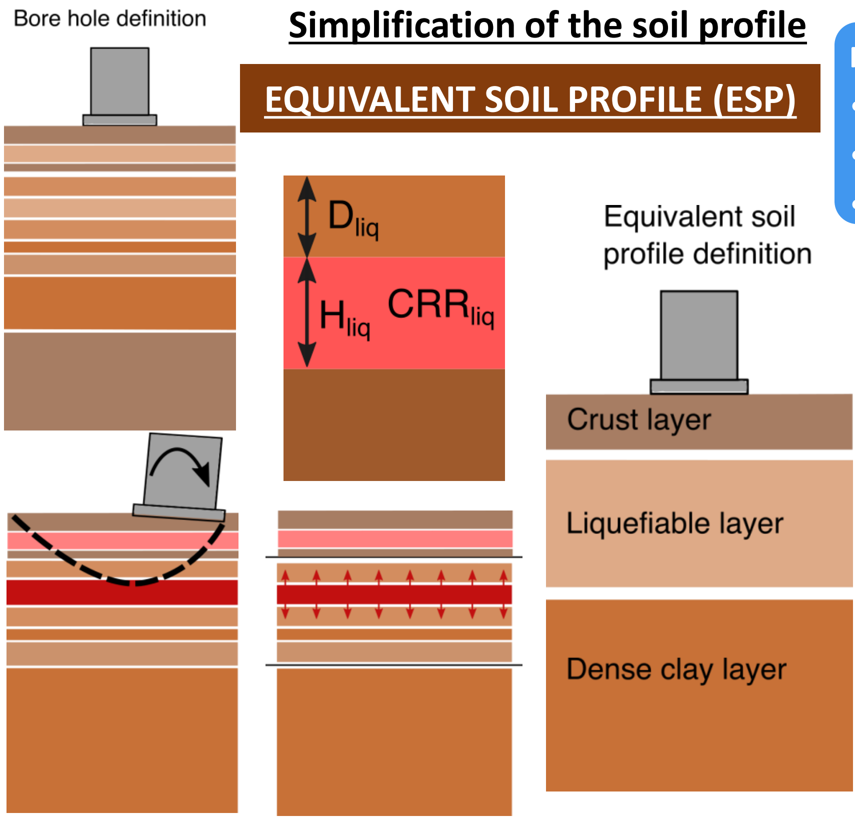


Necessary to define a systematic process of classifying liquefaction resistance of soil profiles, using a standard seismic hazard or independent of seismic hazard?

The two most important are the **thickness of the crust** and the **height of the liquefied (or liquefiable) layer**. These influence building, the characteristics and intensity of ground surface, the manifestation of liquefaction at the surface and the soil stiffness or foundation impedance.

Simplification of the soil profile

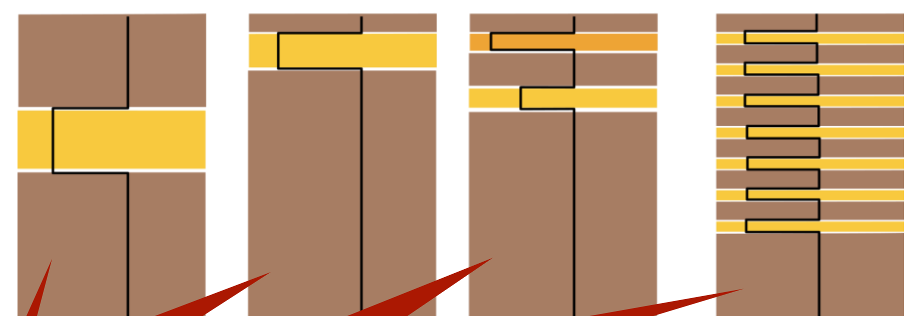
EQUIVALENT SOIL PROFILE (ESP)



Need to answer:

- What level of differential settlement could be expected in the structure?
- What level of shaking could be expected in the structure?
- What level of soil-foundation stiffness could be expected?

CRR profile for LSN = 20 @ PGA = 0.15g



Existing methods provide non-unique soil profiles

And do not account for the presence of the building and the time to liquefaction.

SIMPLIFIED PROCEDURE TO CLASSIFY SOIL PROFILES

It is a <u>simple three layers classification criteria</u>, **hazard-independent** adopted **for performance and loss assessment of buildings on shallow foundations**, using the **height (Hliq)** and **depth to the critical liquefiable layer (Dliq)**, and **average cyclic resistance of the layer for 15 cycles** of uniform load (**CRRn15**).

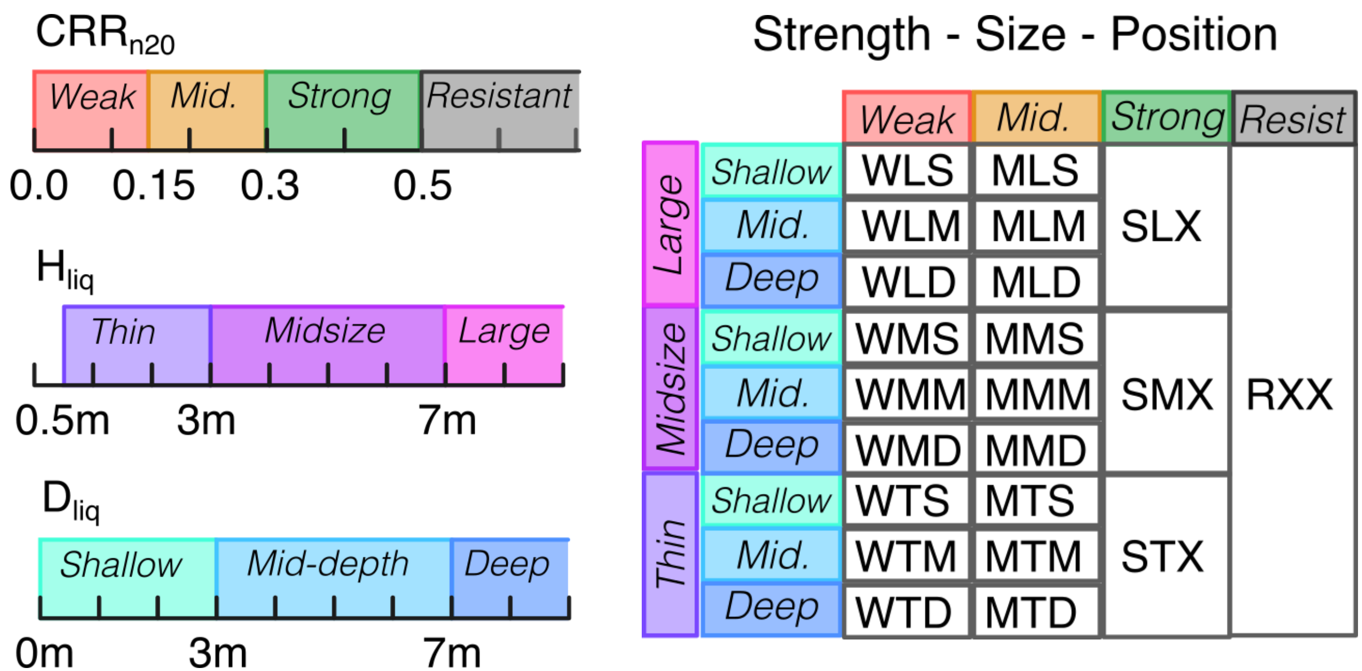
New classification for liquefaction potential

The <u>main advantages</u> of this approach are:

(i) determined from CPT, DMT, SPT seismic waves surveys or borehole data; at the site.

(ii) captures the soil profile behaviour across the full hazard range using just three values;

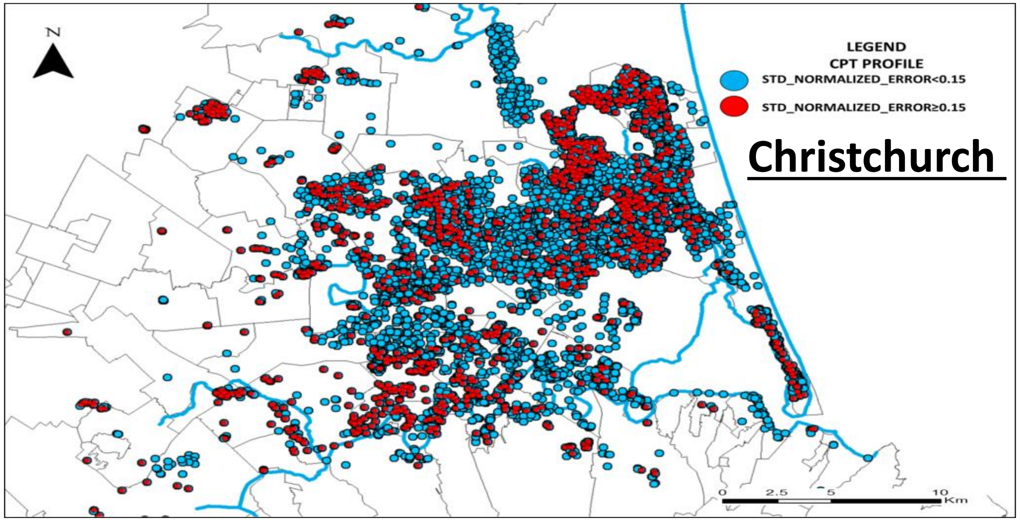
(iii) information is directly related to building performance;



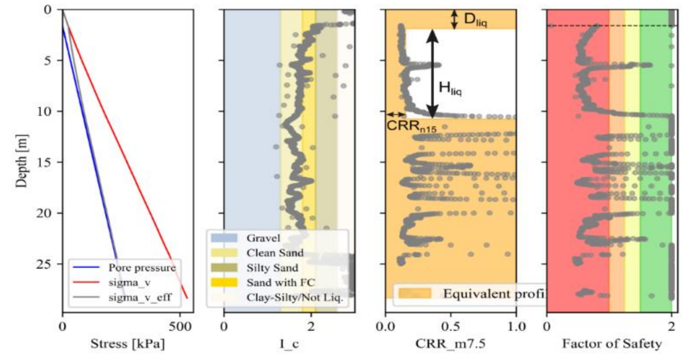
(iv) can capture complex system effects (e.g. vertical pore water flow);

(v) intuitive parameters are used (soil layering vs foundation geometry and hazard level), rather than strains or quality indexes (e.g. LPI or LSN);

(vi) can provide a definition of the profile without knowing the seismic hazard at the site.

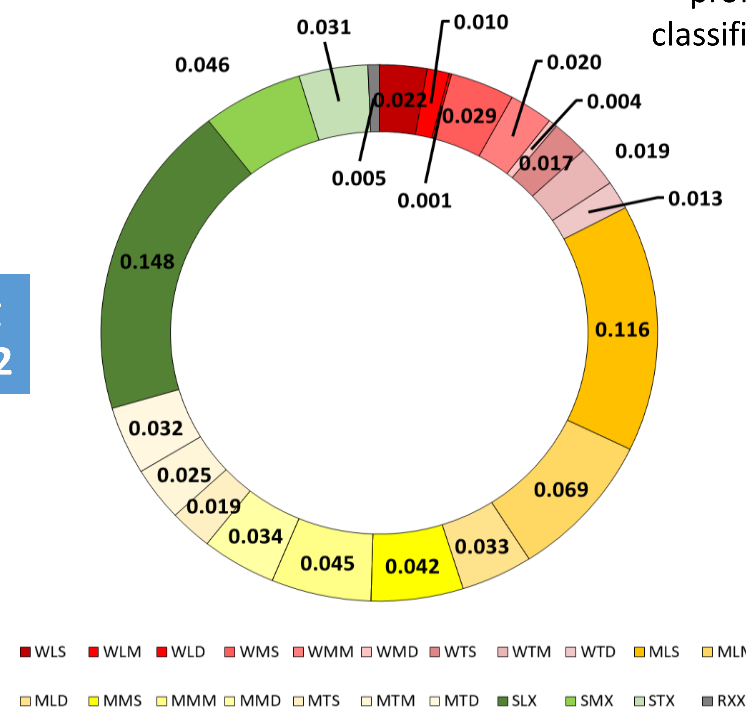


Christchurch



Case study single profile classified

ESP DISTRIBUTION



Strength - Size - Position

		Weak	Mid.	Strong	Resist
Large	Shallow	WLS	MLS	SLX	RXX
	Mid.	WLM	MLM		
	Deep	WLD	MLD		
Midsize	Shallow	WMS	MMS	SMX	
	Mid.	WMM	MMM		
	Deep	WMD	MMD		
Thin	Shallow	WTS	MTS	STX	
	Mid.	WTM	MTM		
	Deep	WTD	MTD		

Connecting WP3 to WP2

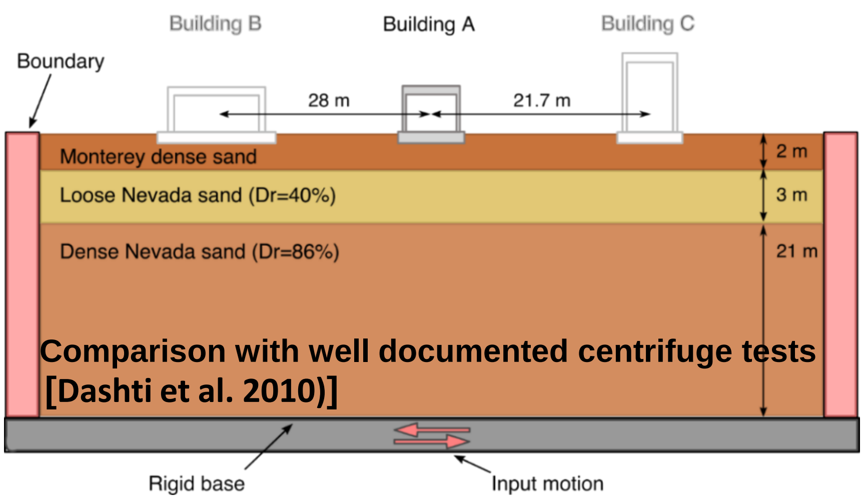
ESP distribution of the 22 classes

ESTIMATION OF SITE RESPONSE AND SOIL-STRUCTURE INTERACTION USING EFFECTIVE STRESS ANALYSIS

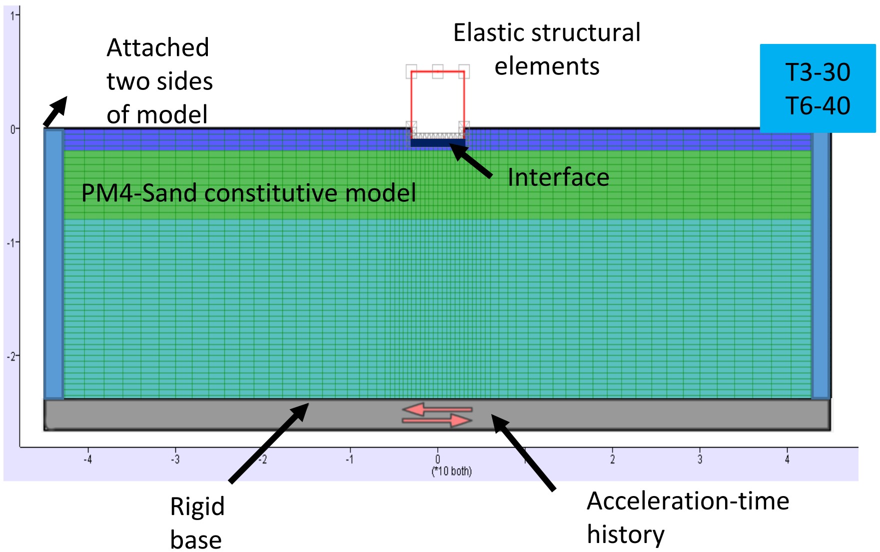
Numerical calibration with centrifuge data of seismic response of buildings on liquefiable soils

The PM4Sand constitutive model (a sand plasticity model for geotechnical earthquake engineering) was implemented in the commercial software, FLAC 8.0:

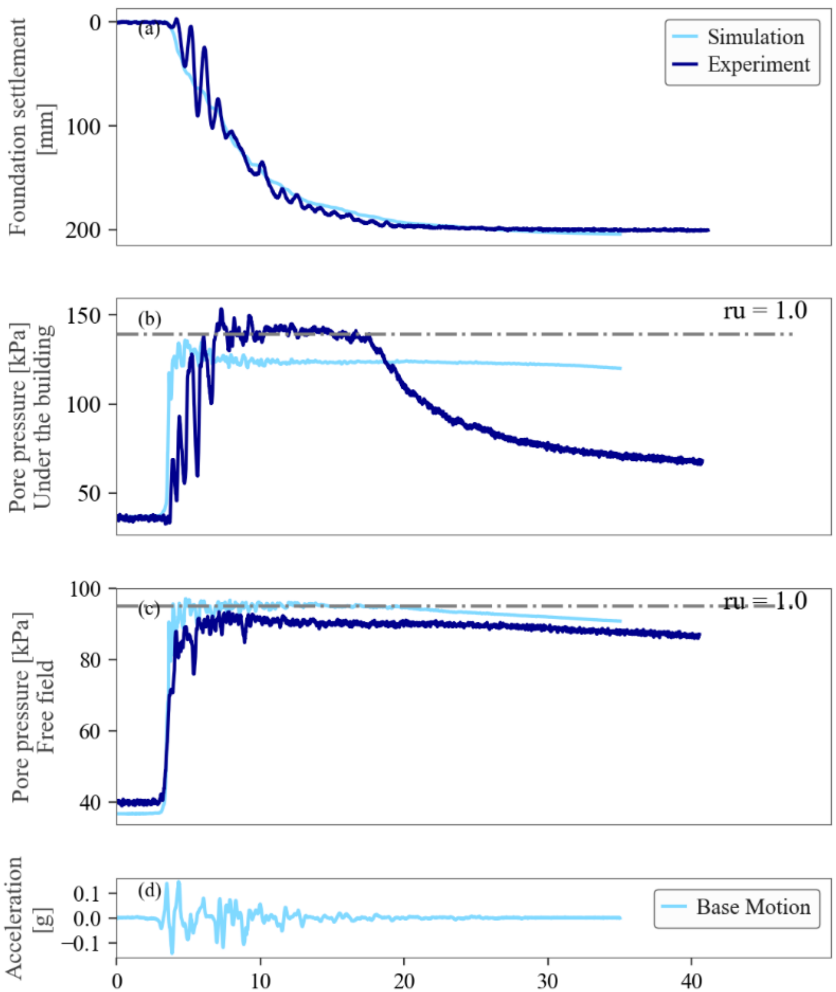
- 2D FLAC models were calibrated to give in nonlinear dynamic SSI effective stress analysis a reasonable behaviour of the response of structures of well documented centrifuge tests :
  - ➢two different soils were modelled, the behaviour being determined by simulated element tests
- 1D FLAC models were calibrated to fit the behaviour derived from SPT-vs-CRR:
  - ➢then random fluctuations were added to represent the dispersion of behaviour seen in real soil;
  - ➢actual behaviour was then back-calculated

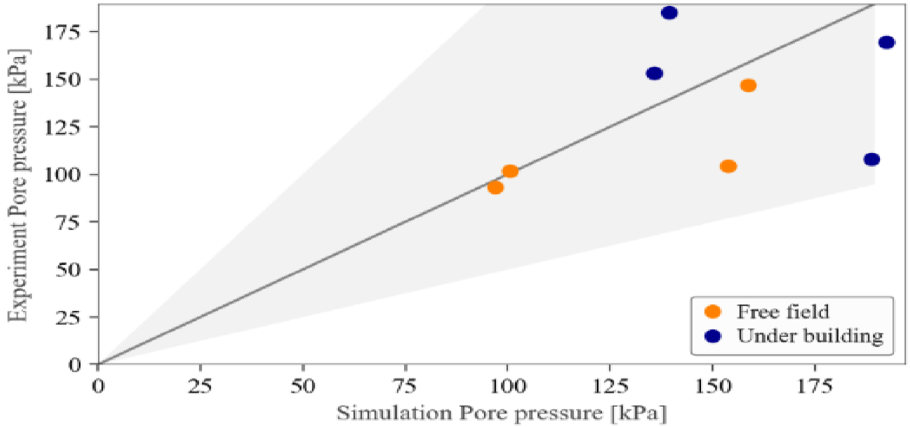


Comparison with well documented centrifuge tests [Dashti et al. 2010)]

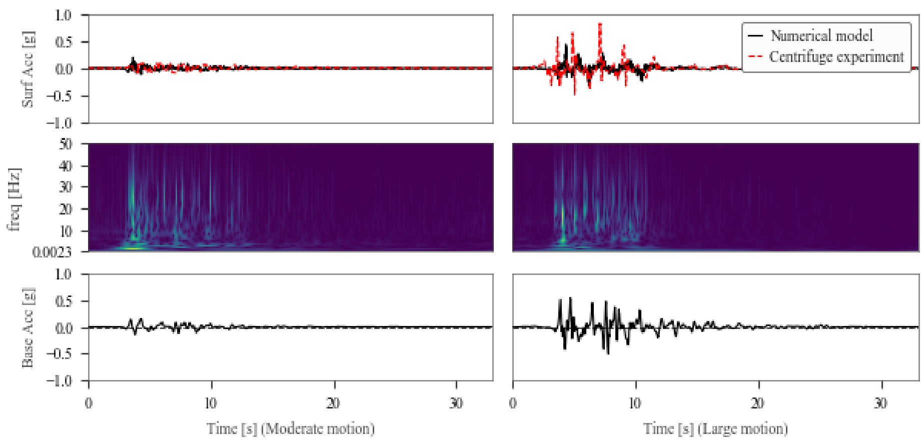


Surface Shaking, pore pressure and Settlements
NUMERICAL CALIBRATION via FLAC

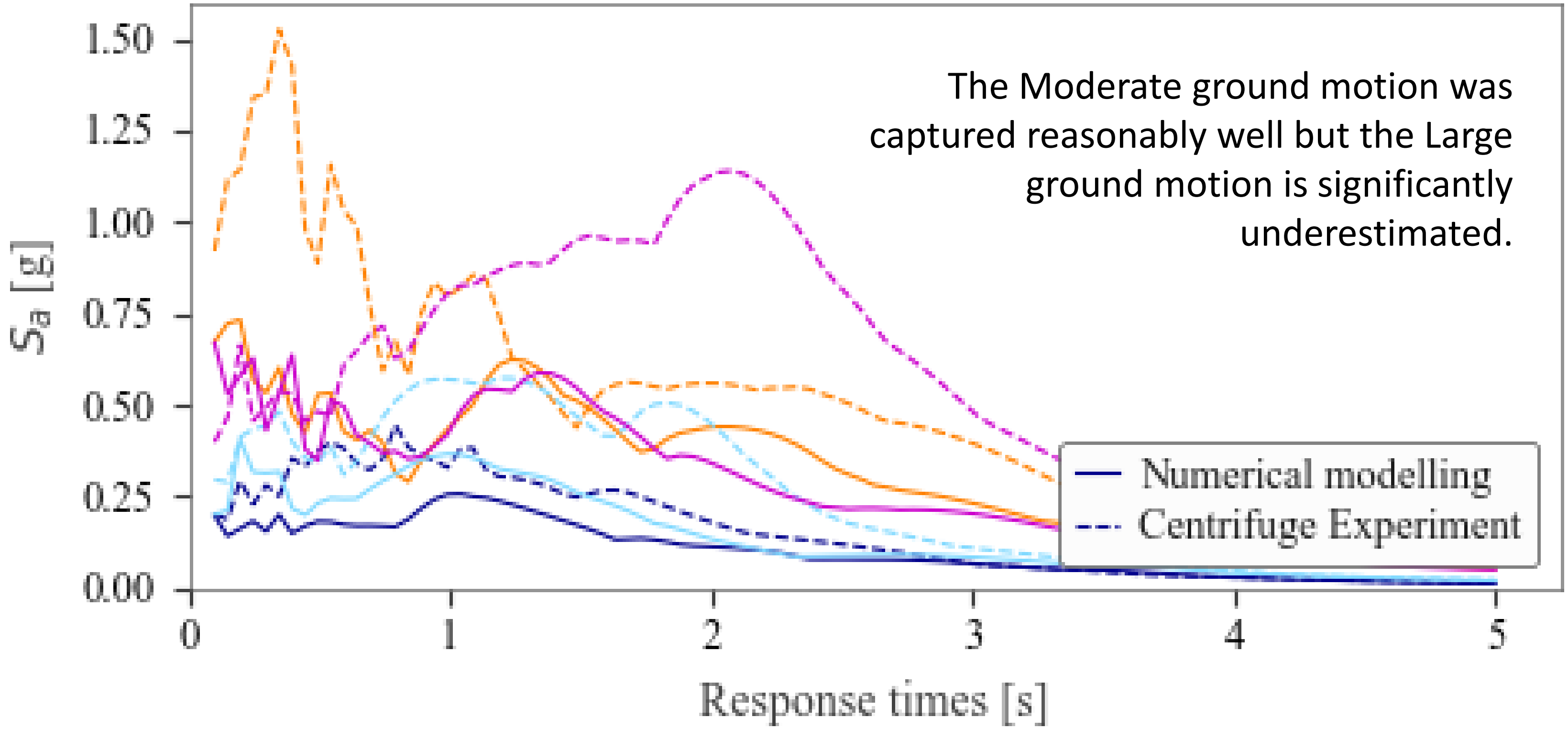




The figure follow how the acceleration signal in the surface (free-field) and the Stockwell transform



Comparison were made between acceleration response spectra (Sa) at surface of the model for the recorded motions in the centrifuge experiment and the calculated in the numerical analyses for both experiments and ground motions.



The Moderate ground motion was captured reasonably well but the Large ground motion is significantly underestimated.

Simplified strain ENERGY-based method (SEBM)

DOES LIQUEFACTION ACT AS A NATURAL ISOLATOR ???

- **SIMPLIFIED PROCEDURE TO ESTIMATE PORE PRESSURE IN FREE FIELD**

Stress-based methods

Seed et al. (1975) and following developments

(…)

Energy-based methods

Millen et al. (2019) – developed in UPorto
Soil Dynamics and Earthquake Engineering

➢ SIMPLIFIED PROCEDURE TO ESTIMATE PORE PRESSURE IN FREE FIELD

Stress-based method

Conversion of an irregular earthquake ground motion to an equivalent number of uniform cycles (N)

Seed et al. (1975) and following developments

$$r_u = \frac{2}{\pi} arcsin\left[\left(\frac{N}{N_L}\right)^{1/2\beta}\right]$$

$$\beta = 0.7$$

Booker et al. (1976)

$$\beta = c_1 FC + c_2 Dr + c_3 CSR + c_4$$

Polito et al. (2008)

$$CSR = a.N^{-b}$$

$$\frac{N_A}{N_B} = \left(\frac{CSR_B}{CSR_A}\right)^{1/b} \Leftrightarrow MSF = \frac{CSR_M}{CSR_{M=7.5}} = \left(\frac{N_{M=7.5}}{N_M}\right)^{b}$$

$$\frac{N_L}{N} = \sum N_{ref} * \left(\frac{CRR}{CSR}\right)^{1/b}$$

$$CSR = |acc_{peaks}| * \frac{\sigma_{v0}}{\sigma'_{v0}} * r_d$$

$$r_d = e^{[f(z)+g(z)*M]}$$ Idriss (1999)

$$f(z) = -1.012 - 1.126 * sin\left(\frac{z}{11.73} + 5.133\right)$$

$$g(z) = 0.106 + 0.118 * sin\left(\frac{z}{11.28} + 5.142\right)$$

The b value needs to be defined! Idriss (1999) proposed 0.34

➢ SIMPLIFIED PROCEDURE TO ESTIMATE PORE PRESSURE IN FREE FIELD

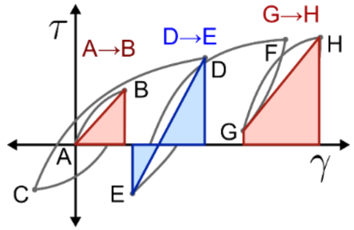
Energy-based method

Millen et al. (2019)

Method based on the conservation of energy

Soil resistance to liquefaction: measured in terms of *normalised cumulative absolute strain energy (NCASE)*

$$NCASE = \sum_{j=0}^{n_{peaks}} |\tau_{av.,j}| \cdot |\gamma_{j+1} - \gamma_j| / \sigma'_{v0}$$



Estimation of earthquake demand: by the *cumulative absolute kinetic energy (CAKE)*

$$\text{CAKE} = \rho \cdot \sum_{i=1}^{n} \Delta(\dot{u}_i \cdot |\dot{u}_i|) = \rho \int \left| \frac{d\dot{u}_i^{\,2}}{dt} \right| dt$$

The corresponding intensity measure is called UKE – unit cumulative absolute kinetic energy.
CAKE is used to provide an exact solution for the NCASE at any depth in a homogenous purely linear elastic soil deposit using the nodal surface energy spectrum (NSES)

Estimation of pore pressure:

Estimated by CAKE

$$r_{u,i} = \sqrt{\frac{NCASE_i}{NCASE_{liq}}} \cdot r_{u,liq}$$

ru_{limit} (for example, 0.98 or 5% of σ'_{v0})

$$NCASE_{liq} = \frac{2 \cdot CSR^2 \cdot \sigma'_{v0} \cdot n_{liq}}{G_i \cdot \left(1 - \frac{CSR}{\sin(\phi_{cv})}\right)} \cdot \kappa$$

k is a calibrating parameter that can be taken equal to 3 for PM4sand model

n_{liq} is the reference number of cycles at liquefaction corresponding to the CSR, for example 15 for CSR_{15}.

➢ Modelling recomendations

Simplified stress based method - SBM

- SBM need a convertion to the equivalente number of cycles, which is dependent on the soil capacity through the CSR_{15} and b value, and limits the efficiency of this method.
- This equivalent conversion procedure has several uncertainties related to r_d equation, and to the estimation of surface acceleration.
- Finally, this equivalent cycle procedure assumes the shear stress to be constant throughout the earthquake, whereas typically shear stresses reduce due to softening of the soil with increased excess pore water pressure.
- In the absence of laboratory tests, the b value should be taken as 0.34 to be consistent with Boulanger et al. (2016) and the pore pressure build-up should use the β value calculated for each specific case using equation from Polito et al. (2008) based on the relative density, cyclic stress ratio and fines content. If no information is available the value of 0.7 proposed by Booker et al. (1976) is a valid option.

Simplified strain energy based method - SEBM

- In the absence of laboratory tests, the SEBM should be used with the normalised strain energy being calculated using this equation

$$NCASE_{liq} = \frac{2 \cdot CSR^2 \cdot \sigma'_{v0} \cdot n_{liq}}{G_i \cdot \left(1 - \frac{CSR}{\sin(\phi_{cv})}\right)} \cdot \kappa$$

ESTIMATING SURFACE GROUND MOTION

Main damages to buildings during earthquakes is shaking damage. Modification to the ground shaking due to liquefaction is very important to quantify building performance.

Ground motions from liquefied deposits have less high frequency content and can have larger displacement demands than their non-liquefied equivalents.

These perceived beneficial effects have even prompted interest in **deliberately using liquefaction to isolate buildings from strong shaking**.

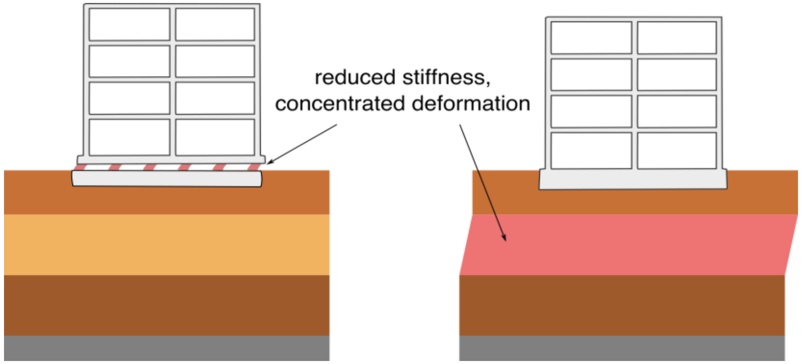
However, liquefaction does not always result in less shaking. Bouckovalas et al. (2016) demonstrated that **liquefaction of the soil can cause an amplification in the seismic shaking especially in lower frequencies which is highly dependent on the depth of the liquefied layer**. *In 2011 Christchurch earthquake post-liquefaction acceleration spikes doubled the size of pre-liquefaction acceleration values.*

Liquefaction causes a reduction in soil stiffness, increase in soil shear strain, and can amplify and reduce particular frequencies of the surface shaking

ESTIMATING SURFACE GROUND MOTION

Conceptually the reduction in stiffness can provide protection to buildings similar to base isolation techniques used within structural engineering and is often referred to as "natural seismic isolation":

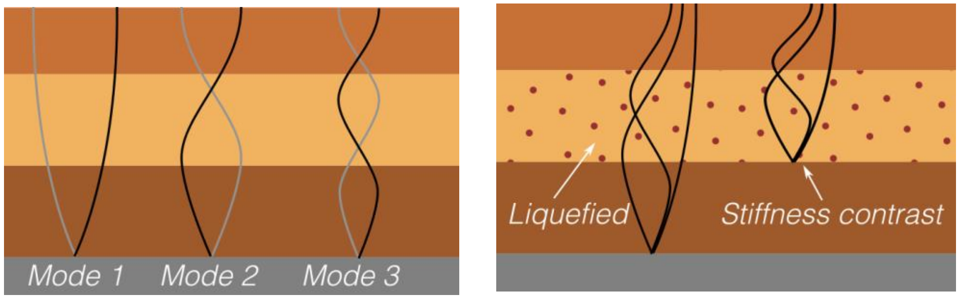
High frequency content reduces, **low frequency content can amplify**



The reduced stiffness lengthens the characteristic site period and means that shear waves dissipate more energy over the same distance because shear wave speeds have reduced, this is particularly evident for small cycle (high frequency) waves. **The energy dissipation per cycle is also increased because the softer soil undergoes larger nonlinear strains and therefore the liquefied layer can act as a high-pass filter.**

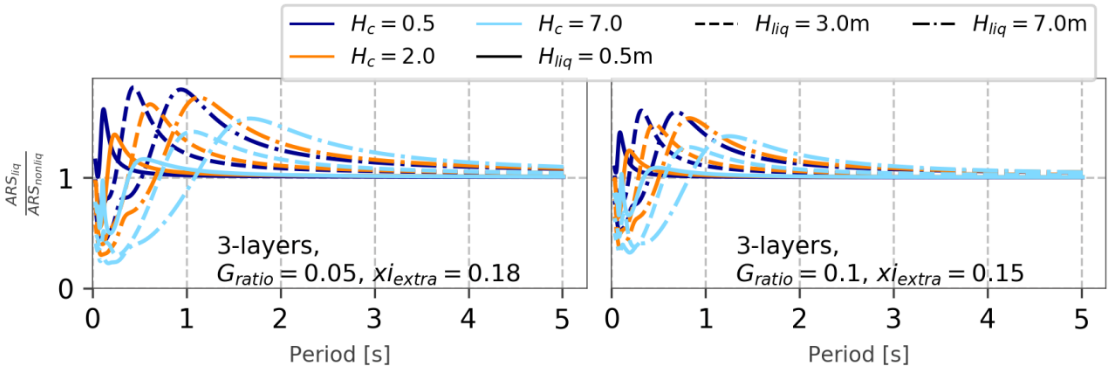
However, not all frequencies are reduced. In some cases, frequencies can be amplified. **When shaking frequencies are close to the fundamental frequency of the deposit the upward propagating wave reflects off the surface and superimposes forming standing wave that increases the surface shaking amplitude**

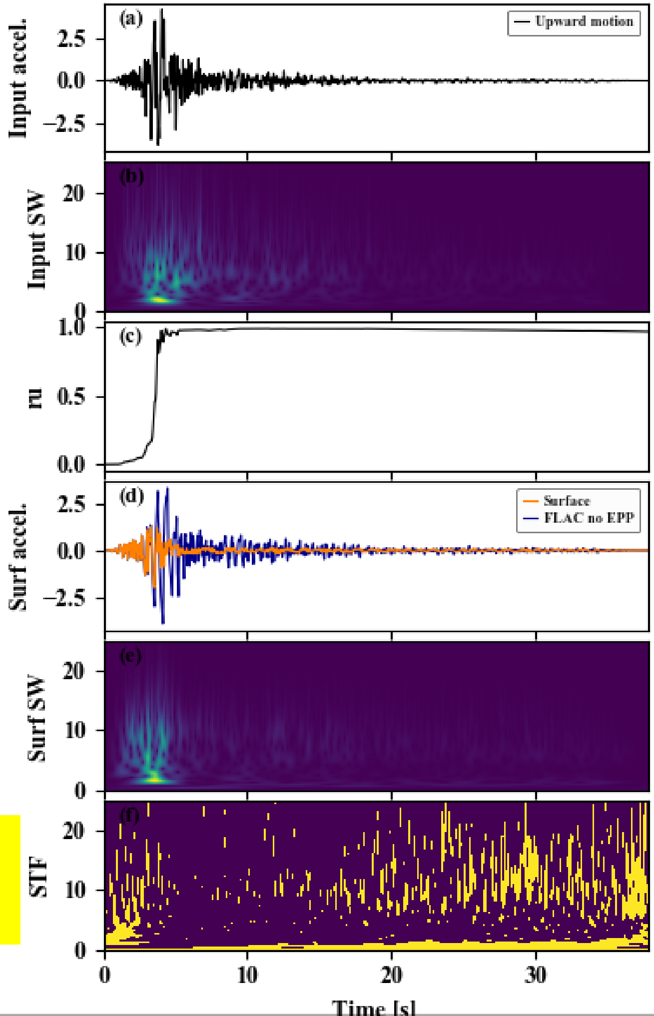
Frequency content modification depend on soil stratigraphy and location and size of liquefaction



The effects of liquefaction happen dramatically at the time of liquefaction

Ratio of surface spectral acceleration between liquefied and non-liquefied deposits for different stratigraphy:





FLAC 1D site response, with and without pore pressure build-up (sharp change in behaviour r_u>0.9)

Stockwell Transfer Functions (STF)

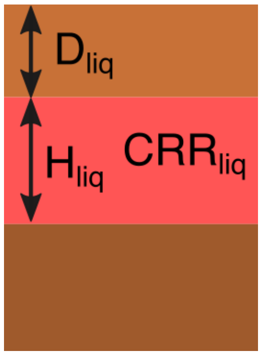
Ratio of the surface and input STF: notice the yellow reduces after liquefaction

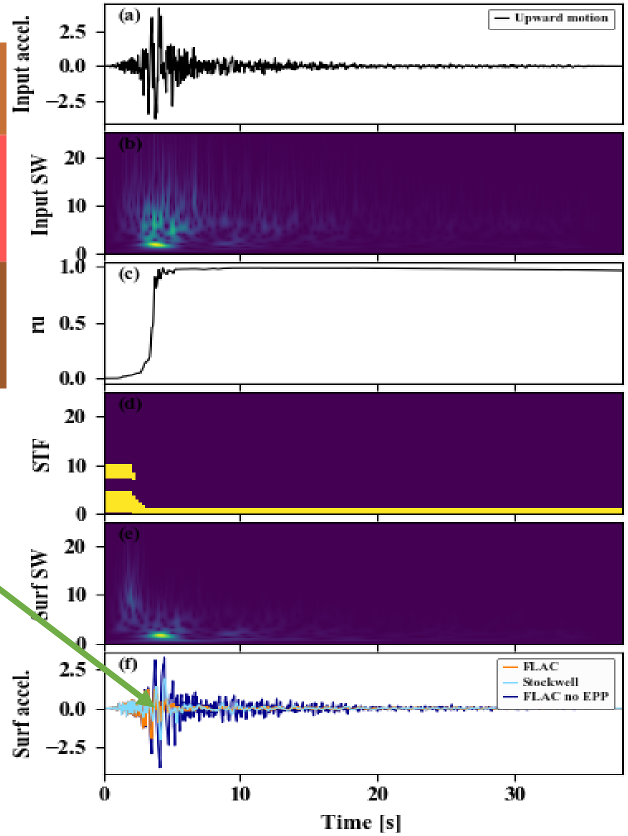
THE STOCKWELL TRANSFER FUNCTION METHOD

In equivalent linear analysis the transfer function between the upward motion and surface is constant for the whole ground motion. **However, in the event of liquefaction, the dramatic reduction in stiffness and increased energy dissipation makes the assumption of a constant transfer function invalid.** Several analysis from the 500 analyses, using the pore pressure induction method illustrate the influence of liquefaction on the surface acceleration and transfer function.

The surface acceleration of the FLAC analysis with pore pressure build-up is shown in comparison to the same analysis where excess pore pressure was prevented by setting the water bulk modulus to zero.

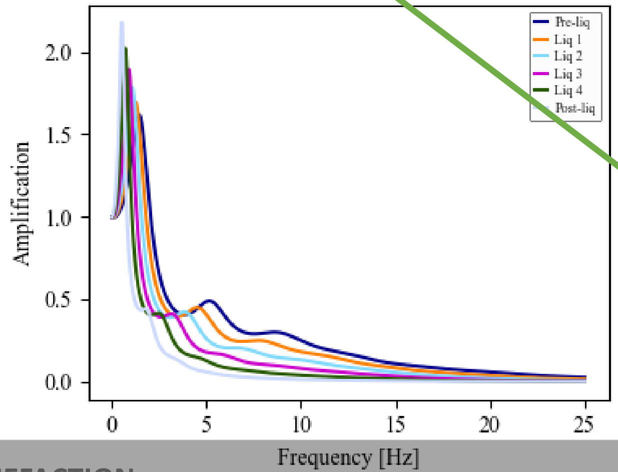
The reduction in acceleration amplitude due to pore pressure build-up is dramatic.





Pseudo site response

Estimate time of liquefaction, then develop transfer functions at times prior to liquefaction by assuming reduced stiffness and increased damping in liquefied layer.



SIMPLIFIED PROCEDURE TO ESTIMATE SURFACE GROUND MOTION

COMPARISON OF METHODS / THE STOCKWELL TRANSFER FUNCTION METHOD

From the results with excess pore pressure using Stockwell method are **in close agreement**, except for the additional high frequency content in the FLAC analysis prior to liquefaction. Meanwhile the FLAC with no excess pore pressure and the equivalent linear analyses are also in close agreement.

This suggests that at least the Stockwell transfer function method correctly mimics the influence of liquefaction with respect to the response of the two SDOFs.

The ability to rapidly assess the impact of time of liquefaction and of strength and stiffness degradation is one of the major advantages of this method and is a useful benchmark for more advanced analyses.

SIMPLIFIED PROCEDURE TO ESTIMATE SETTLEMENTS OF BUILDINGS

Existing methodologies for estimating settlements

Karamitros et.al (2013)

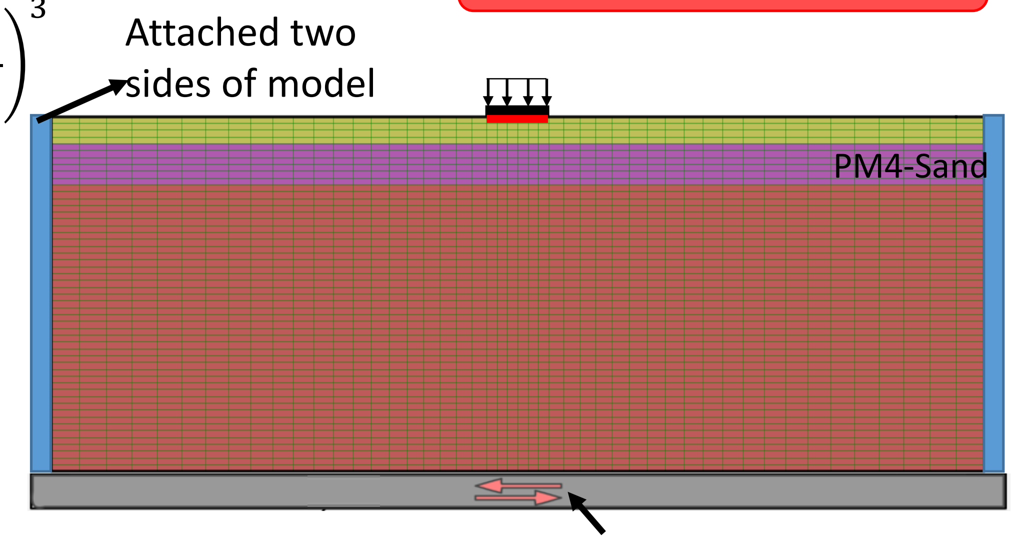
$$Sett_{dyn} = c \cdot a_{max} \cdot T^2 \cdot N \left(\frac{Z_{liq}}{B}\right)^{1.5} \left(\frac{1}{FS_{deg}}\right)^3$$

$$a_{max} \cdot T^2 \cdot N = \pi^2 \int_{t=0}^{N \cdot T} |v(t)| dt$$

$$\varphi_{deg} = \tan^{-1}[(1-U)\tan\varphi_0]$$

$$U = \frac{A}{1+\frac{\Delta\sigma_{v,c}}{\sigma'_{vo,c}}}$$

Flac (Itasca, 2016)



Bray and Macedo (2017)

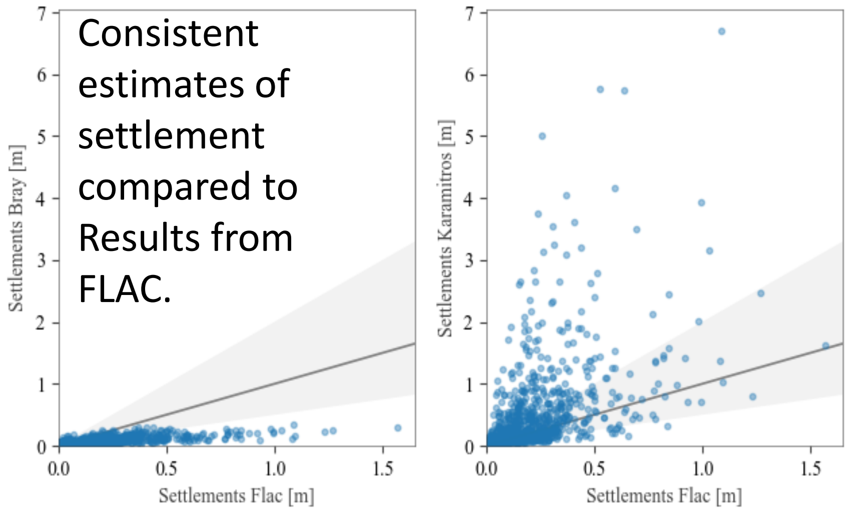
$$\ln(D_S) = c1 + 4.59 \cdot \ln(Q) - 0.42 \cdot \ln(Q)^2 + c2 \cdot LBS + 0.58 \cdot \ln\left(\tanh(H_L)\right) - 0.02 \cdot B + 0.84 \cdot \ln(CAVdp) + 0.41 \cdot \ln(S_a) + \varepsilon$$

$$CAVdp = \sum_{i=1}^{N}\left(H(x)\int_{i-1}^{i} |a(t)|\, dt\right) \qquad LBS = \int W \cdot \frac{\varepsilon_{shear}}{z} dz$$

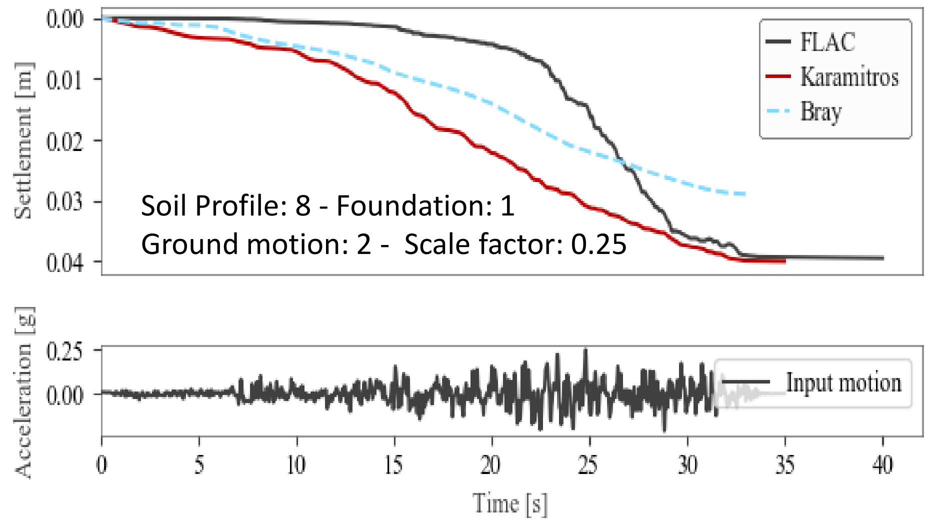
Checking existing methodologies

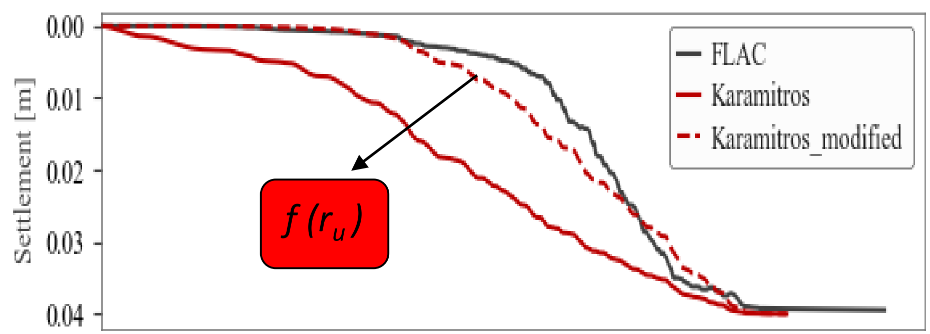
1350 analyses

- 9 soil profiles
- 5 foundations types
- 10 ground motions
- 3 scale factors



Consistent estimates of settlement compared to Results from FLAC.





A new methodology (**Karamitros modified**) was developed in UPorto, **consisting in multiply the total settlement from the original equation by a "weight" that depends on the Pore pressure ratio (Ru) time-series.**

$$\rho_{dyn,i} = c \cdot \pi^2 \cdot \int_{t=0}^{N\cdot T} |v(t)|dt \cdot \left(\frac{Z_{liq}}{B}\right)^{1.5} \cdot \left(\frac{1}{FS_{deg}}\right)^3 \cdot \frac{\sum_0^i |v_i| \cdot r_{u,i}}{\sum_0^{N\cdot T} |v_i| \cdot r_{u,i}}$$

SIMPLIFIED PROCEDURE TO ESTIMATE SETTLEMENTS OF BUILDINGS

ON THE METHODS

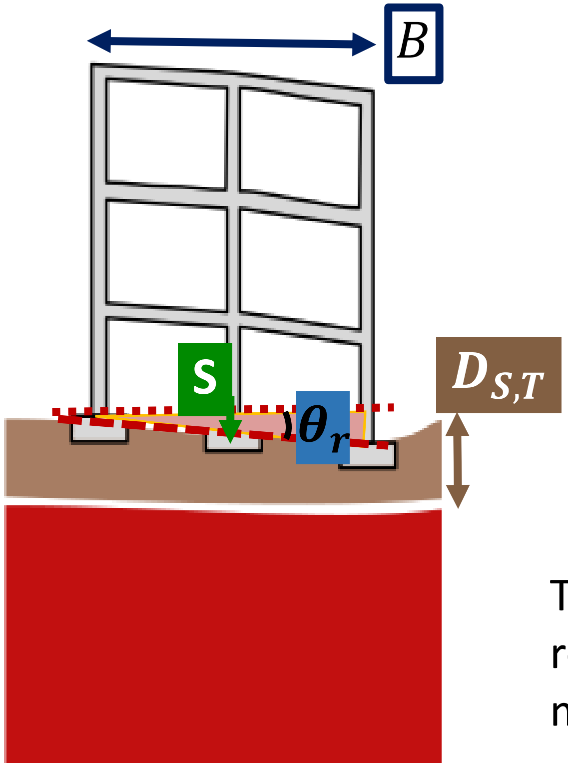
Karamitros et al. (2013) and Bray and Macedo (2017) methodologies, provided viable options for efficiently assessing the total dynamic settlement of a building compared with nonlinear effective stress numerical calculations results (1350 analyses).

The numerical model was validated against centrifuge experimental results (Dashti et al, 2010). This method captures more of the response than the analytical methods as the shear demand, site response, water flow and soil-structure interaction are all directly modelled, but it requires a **high computational effort** (approximately 3 hours for a 40 second ground motion and a 2D foundation-only model) and

therefore, 2D modelling is not justifiable for vulnerability analysis unless the building is deemed critical and susceptible to liquefaction.

Simplified empirical model for estimating **residual tilt**

For structures with isolated footings, the imposed settlement time series S(t) was applied at each constrained node, in order to take into account the liquefaction effects and calculate the building damage. The time series was pre-calculated using one of the methods to calculate settlements and passed to the Opensees model.



In order to include the effects of soil heterogeneity, **at each footing the settlement time series was multiplied by a constant coefficient** **based in Bullock et al. (2019) simplified empirical model for estimating residual tilt.**

Bullock et al. (2019) proposed an empirical model for residual tilt (**θ_r**) based solely on case history observations. This model **depends on the width of the mat foundation (*B*), the thickness of the non-liquefiable crust ($D_{S,T}$) and the average settlement experienced by the foundation (S).**

$$ln(\theta)_r = a_1 \cdot ln(S) + a_2 \cdot ln(B) + a_3 D_{S,T} + \varepsilon_{r^e}$$

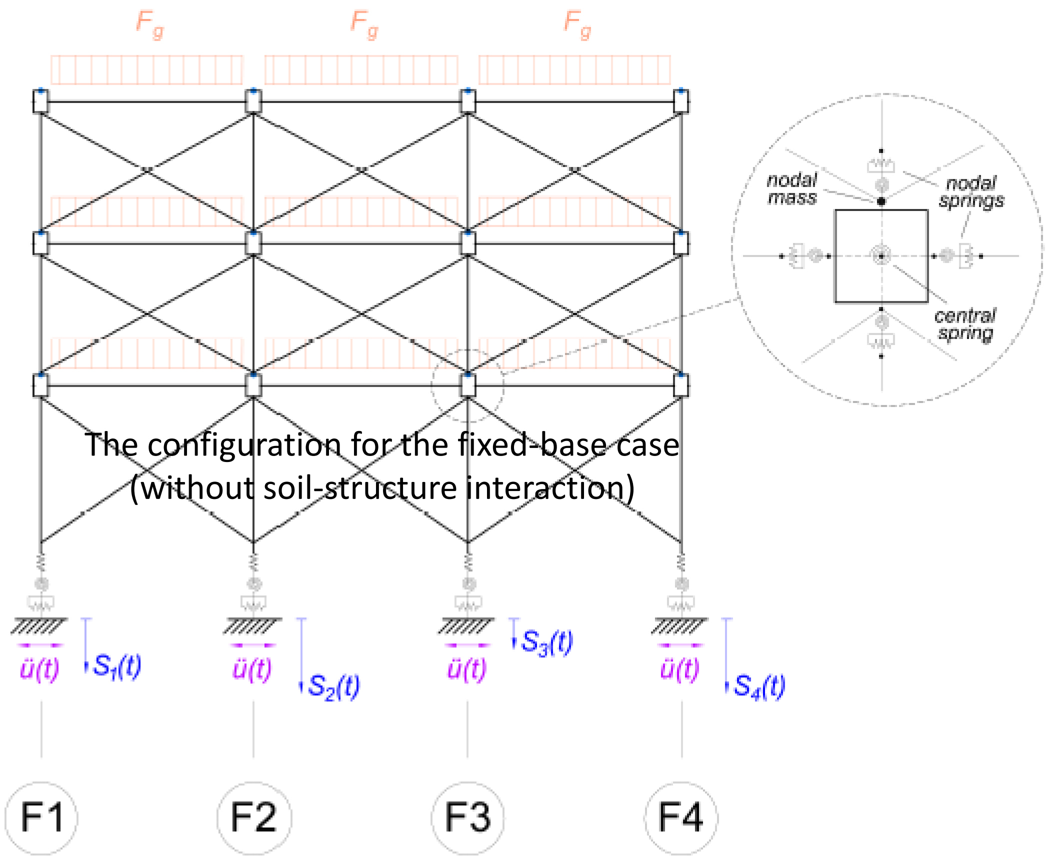
The constant coefficient at each footing depends on the estimated residual tilt and *xfoot* is the distance from the footing axis to the middle of the structure.

$$coeff = \left[\frac{S + x_{foot} \cdot \tan\theta_r}{S}\right] \cdot S$$

Structural model - Foundation settlements

For soil-foundation configurations, foundations were modelled in OpenSees as infinitely rigid in the two displacement directions and in the rotational component. Nodes corresponding to the foundation level were constrained in the three components.

For the structures supported by the non-linear spring-damper system at the footings, a nodal mass corresponding to a half the vertical distributed load acting on the tributary span of the ground floor was placed in correspondence of each footing at the node between the spring-damper system and the column. The remaining 50% of the load was supposed to be directly transmitted to the ground between the footings and was not accounted in the structural analysis.



The configuration for the fixed-base case (without soil-structure interaction)

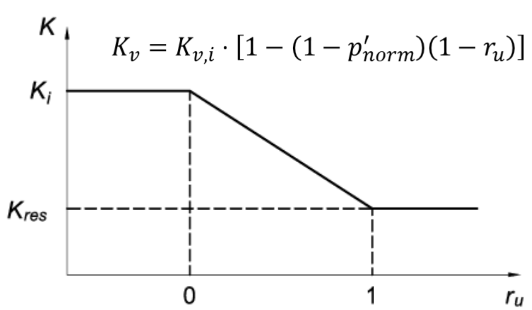
SETTLEMENT

Imposed settlement at the base

- FLAC analysis
- Simplified methods (Karamitros et al. 2013)

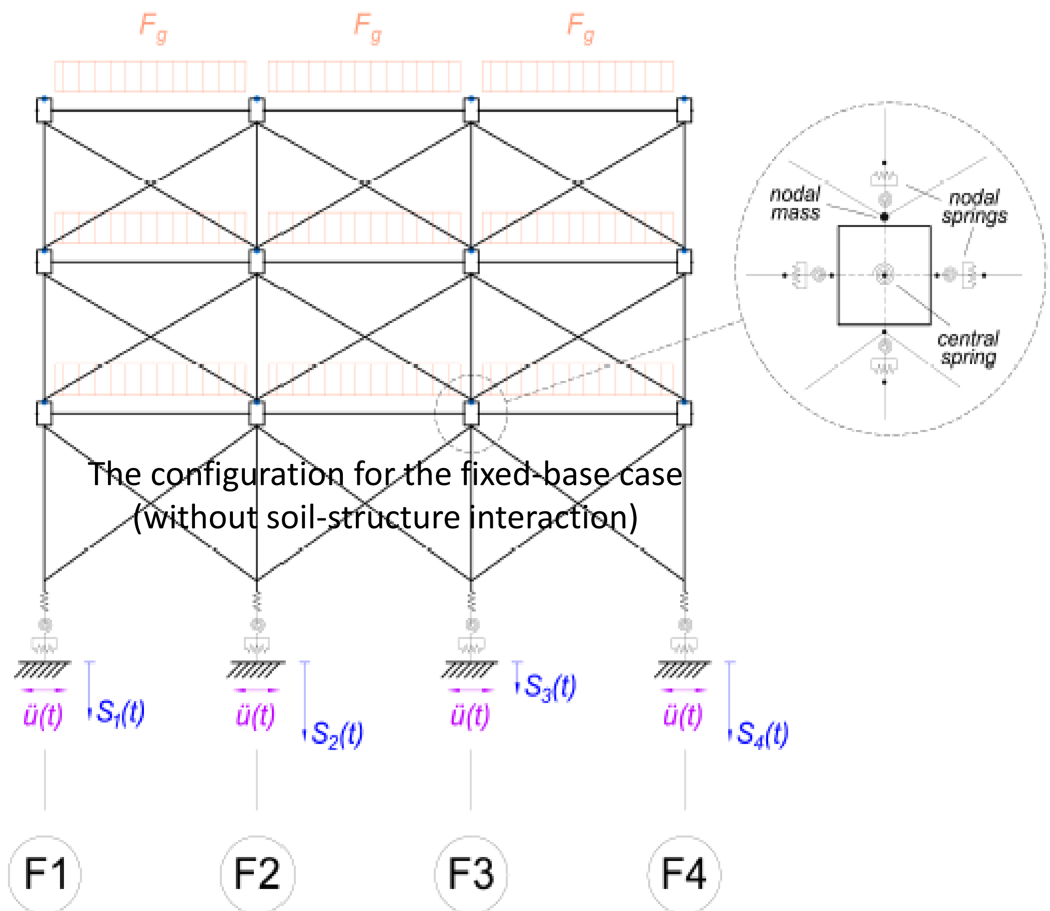
Spring stiffness reduction

- FLAC analysis
- Simplified methods (Karatzia et al. 2017)



The two-dimensional beam-column joints were modelled as parallelogram-shaped shear panels (rotational springs) with adjacent elements connected to their mid-points

Structural model - Foundation settlements



The configuration for the fixed-base case (without soil-structure interaction)

Each beam or column was connected to the shear panel through a shear and a rotational.
The system composed by the shear panel and the four spring elements at the external nodes was able to reproduce the nonlinear response of the structure under monotonic and cyclic strain.

The central rotational spring was modelled with a hysteretic material, with pinching of force and deformation, damage due to ductility and energy, and degraded unloading stiffness.

The external rotational springs were modelled with a material that simulates a modified deterioration model with peak-oriented hysteretic response. The strength and stiffness associated to these materials are function of the physical characteristics of the corresponding elements sections (beams or columns), that were determined in the design phase, where the reinforcement of the structural elements was calculated.

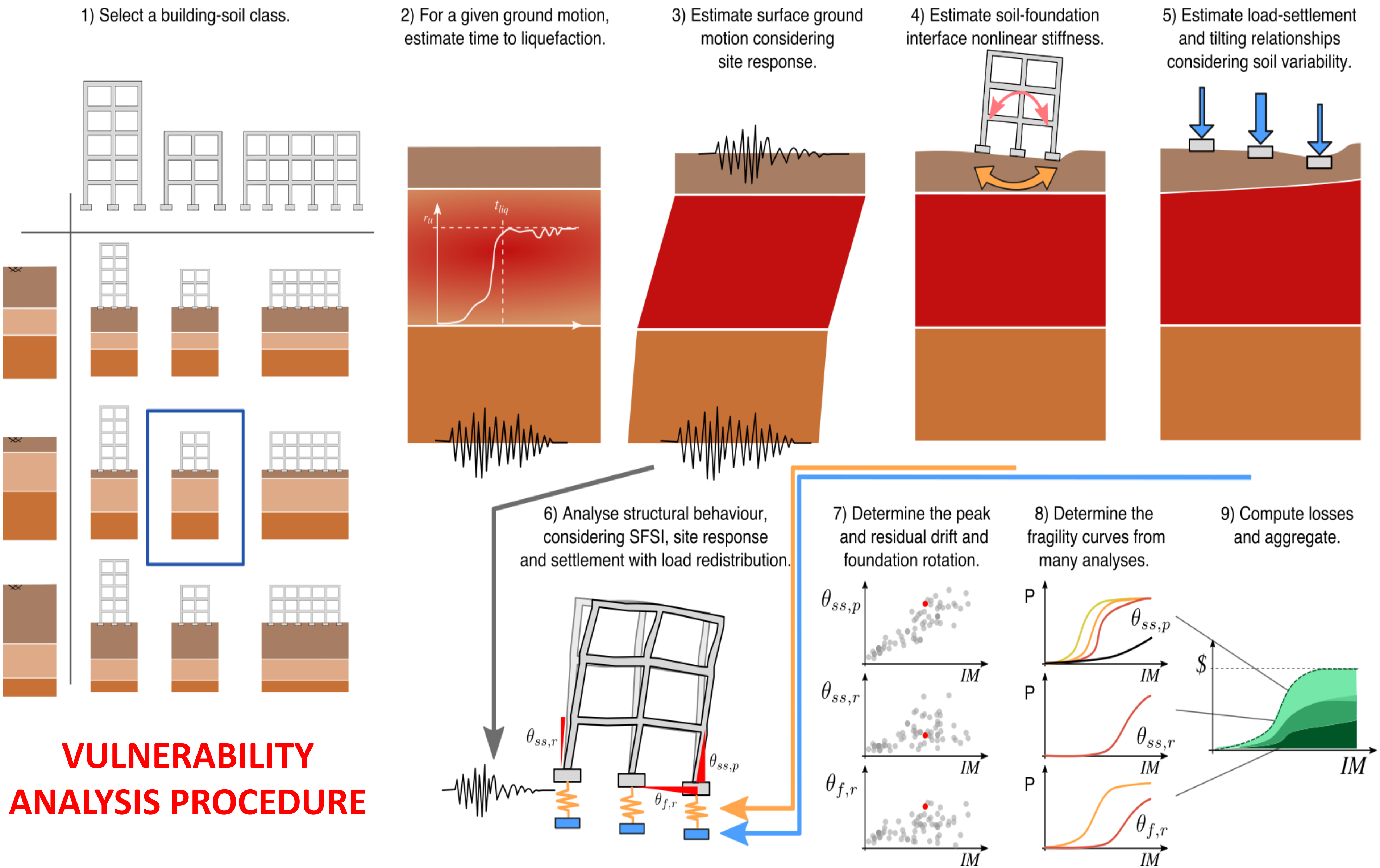
Models with and without masonry infills were analysed.

The elements used were nonlinear truss elements that were assigned a nonlinear stress-strain material model simulating the infill behaviour.

Maximum strength was assumed to be reached at an inter-storey drift of 0.2%.

The lateral displacement of each infill was transformed into the diagonal displacement for the subsequent definition of the strain of the strut.

WP3 - STRUCTURAL LIQUEFACTION RESILIENCE & VULNERABILITY ASSESSMENT METHODOLOGIES



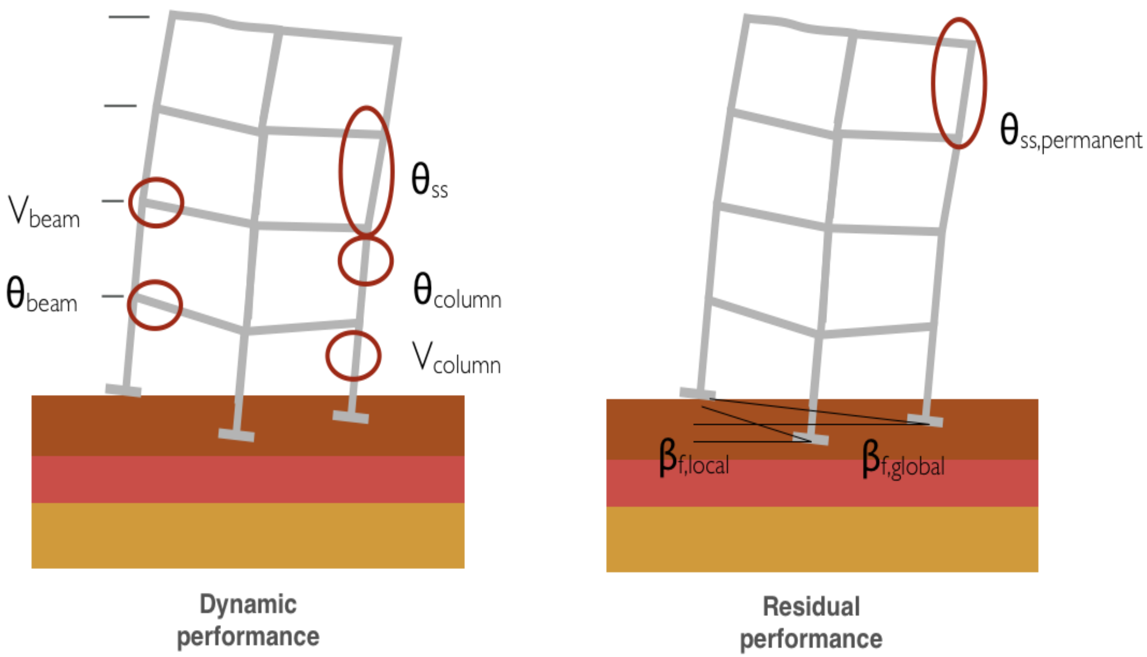
VULNERABILITY ANALYSIS PROCEDURE

THE EFFECTS OF LIQUEFACTION-INDUCED GROUND DEFORMATIONS ON THE PERFORMANCE OF BUILDINGS

It is noted that, ***for columns***, exceedance of limit values of ***shear force*** or ultimate **chord rotation** was considered ***collapse***, while ***for beams*****,** the exceedance of limit values of ***shear force or ultimate chord rotation in a whole storey*** was considered ***collapse***.

For the ***interstorey drift***, ***damage states*** were defined for thresholds of ***0.5%, 1%, 2% and 3%****,* and above 3% while ***above 5%*** was considered ***collapse***.

<u>**Residual performance parameters**</u> involve maximum residual interstorey drift $\boldsymbol{\theta_{ss,r}}$ and residual rotation of the foundation $\boldsymbol{\beta_{f,r}}$ both at the local and global levels.



Dynamic performance parameters and residual performance parameters used to quantify the performance of a building

METHODS TO QUANTIFY VULNERABILITY

The **vulnerability assessment process** requires the building performance to be **evaluated against hazard intensity**, typically with **fragility curves** or **vulnerability curves**.

- The **hazard intensity** is set **using** the ground motion **intensity measures** previously discussed; and,
- The **performance** is obtained by **simulating the building-soil system response** and **calculating** the **performance criteria** referred previously (**collapse, peak local response, peak and residual interstorey drift, and foundation tilt**).

Building damage and performance under liquefaction-induced ground deformations are a direct function of :

- the **properties of the structural system**; but also of,
- the **type of foundation**.

With respect to the structural system, the exposure model and the Building Class Information Model (BCIM) are expected to provide the necessary data for the simulations that need to be performed for a regional seismic risk and loss assessment.

In relation to the **type of foundation** detailed information is not expected to be available and will be very difficult to collect for a large portfolio of buildings.

Therefore, assumptions need to be made and an adequate model for the uncertainty of the foundation system needs to be added to the BCIM model previously described.

COMPUTATION OF LOSSES

The **Pacific Earthquake Engineering Research Centre (PEER)** methodology was developed to answer the need for **communicating seismic risk to stakeholders** involving metrics that reflect seismic consequences, as it allows for the quantification, in probabilistic terms, of different decision variables (DVs) such as **monetary losses, repair time or number of fatalities.**

The basis of the PEER methodology lies in the probabilistic characterization of several performance metrics along with the multiple sources of uncertainty that are inherent to seismic assessment (e.ghazard, the ground motions, the modelling and knowledge-based uncertainties of the building components and properties). The PEER methodology can be summarized by an equation representing the rate of a certain DV exceeding a value *dv* :

$$\lambda(\mathrm{DV}>\mathrm{dv}) = \int_{\mathrm{IM}}\int_{\mathrm{EDP}}\int_{\mathrm{DM}} G(\mathrm{DV}|\mathrm{DM}) \cdot |dG(\mathrm{DM}|\mathrm{EDP})| \cdot |dG(\mathrm{EDP}|\mathrm{IM})| \cdot |d\lambda(\mathrm{IM})|$$

where DM is a damage measure, generally discretised into several damage states, EDP represents measure of the structural response that can be correlated with DM, IM is a ground motion intensity measure and G(·) is the complementary cumulative distribution function (…)

COMPUTATION OF LOSSES

ESP ⟶ STX

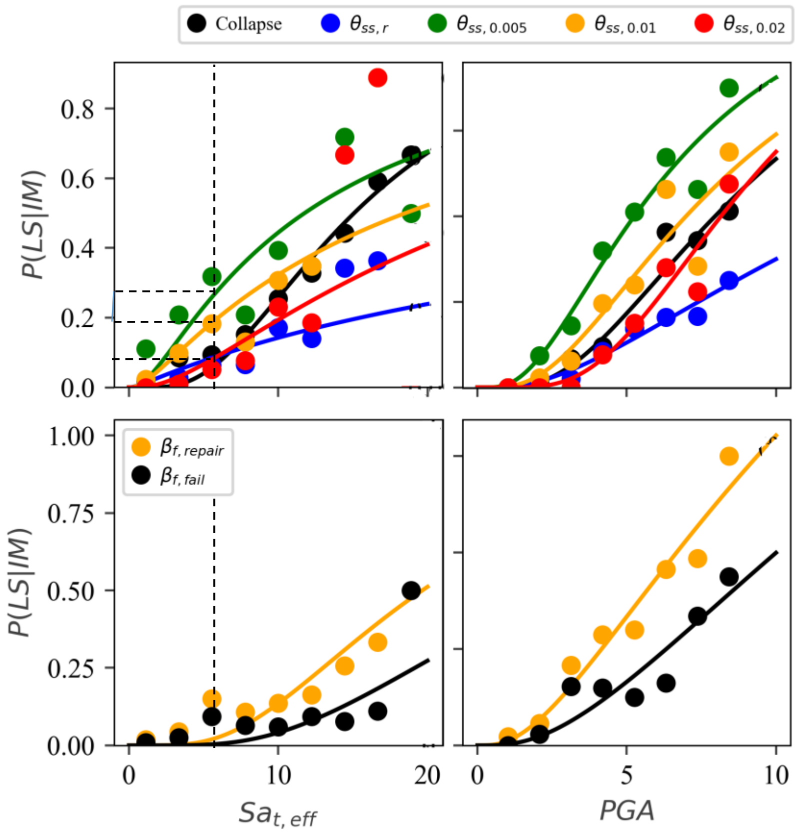
LOSS CALCULATION

EXAMPLE CASE STUDY

Fragility curves were developed for an intensity measure of spectral acceleration at one second (the loss calculations are demonstrated for a spectral acceleration of 0.57s.)

The building losses are based on tables of losses vs peak drift for different number of storeys, building types and building…

The total loss (L_{total}) associated to this hazard level were calculated considering the combined performance probabilities (collapse and residual drift), and where the total loss was 28%.



Obtaining probabilities from fragility curves

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MINISTÉRIO DA CIÊNCIA, TECNOLOGIA E ENSINO SUPERIOR

REPÚBLICA PORTUGUESA

This work is part of the LIQUEFACT project (Assessment and mitigation of liquefaction potential across Europe: a holistic approach to protect structures/infrastructures for improved resilience to earthquake-induced liquefaction disasters), receiving funding from the European Union's Horizon 2020 research and innovation programme under grant agreement No GAP-700748.