Characterization of Gravelly Soils and its Uncertainty in Strength Evaluation

Presented by

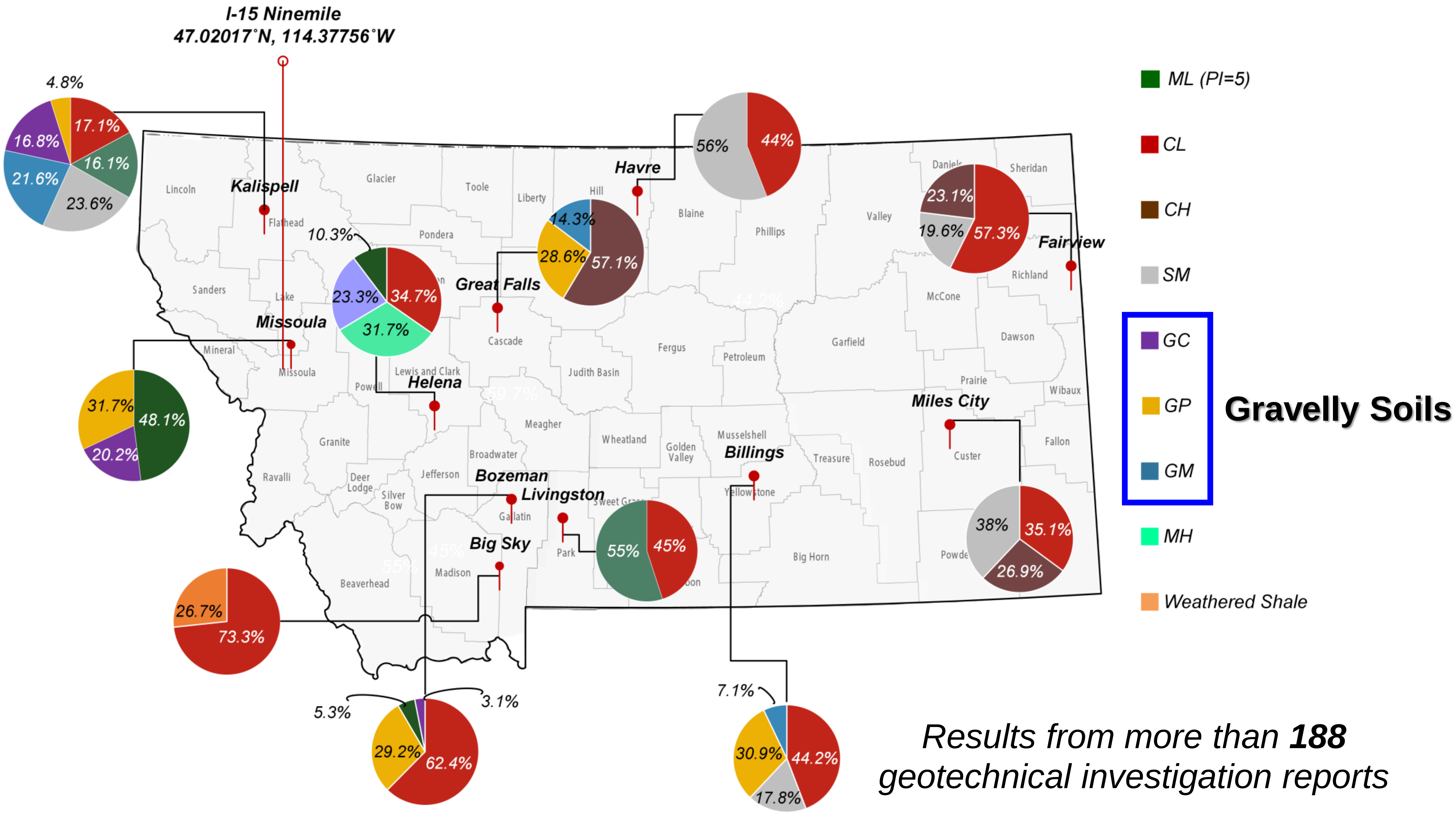
Amalesh Jana, Assistant Professor
and
Gurusamy Kalaiselvan, Ph.D. Student

Civil Engineering Department, Montana State University

Outline of this Presentation

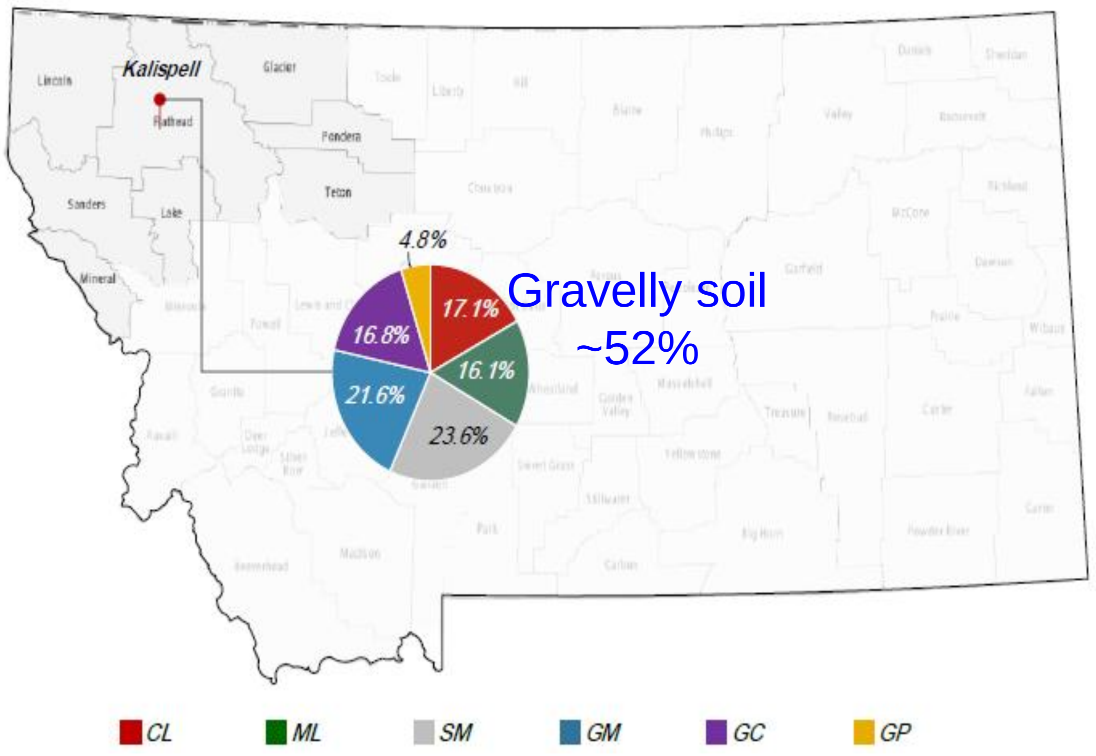
- Soil Conditions in Montana
- Challenge With Gravelly Soils
- Conventional in-situ Penetration Tests
- Strength Estimation of Gravelly Soils
- Seismicity In Montana
- Gravel Liquefaction and Its Effects on Infrastructure
- Example Case Study: Borah Peak Earthquake
- Concluding Remarks

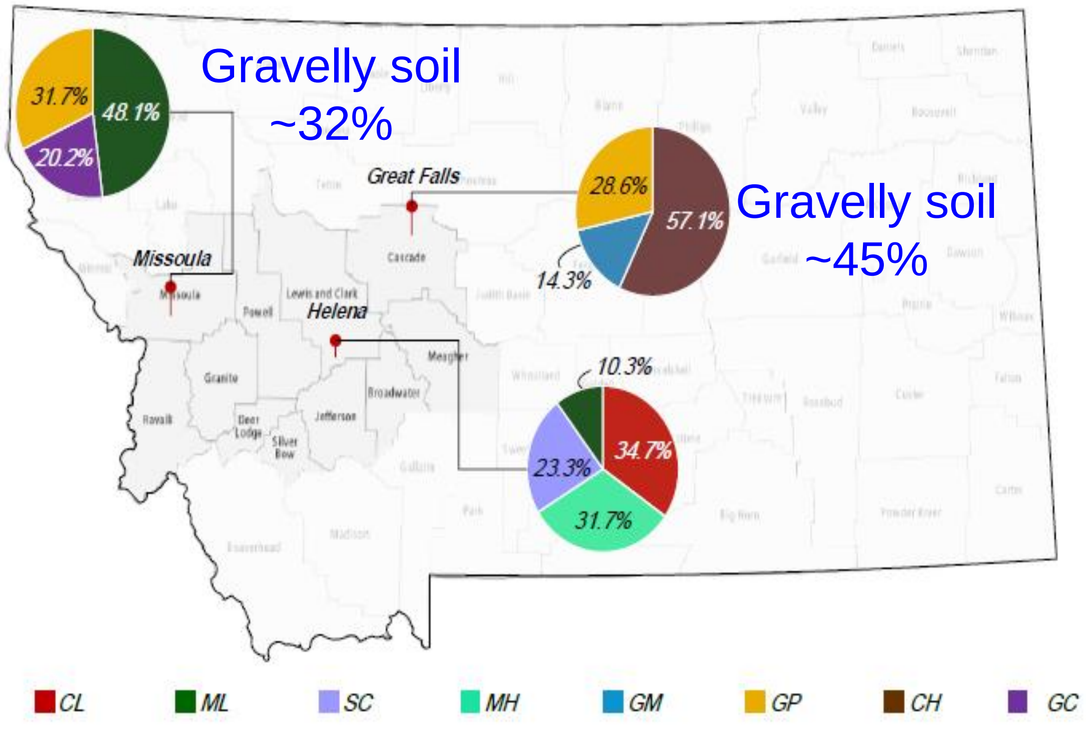
Motivation and Objective: Soil Conditions in Montana



*Results from more than **188** geotechnical investigation reports*

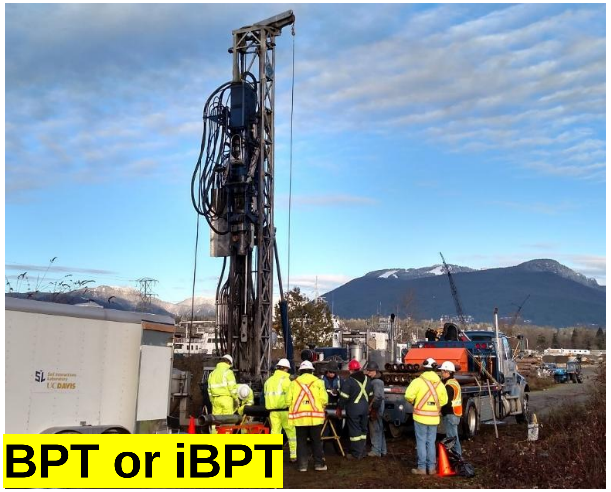
Motivation and Objective: Soil Conditions in Montana

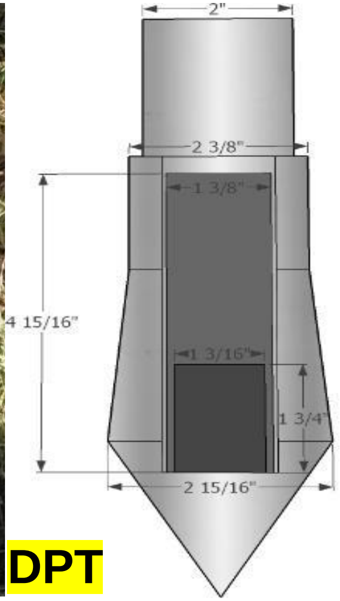




The average soil profiles in different regions have been frequently observed to contain **Gravelly** soils with Silt and Clay contents

Challenge With Gravelly Soils: Conventional in-situ Penetration Tests



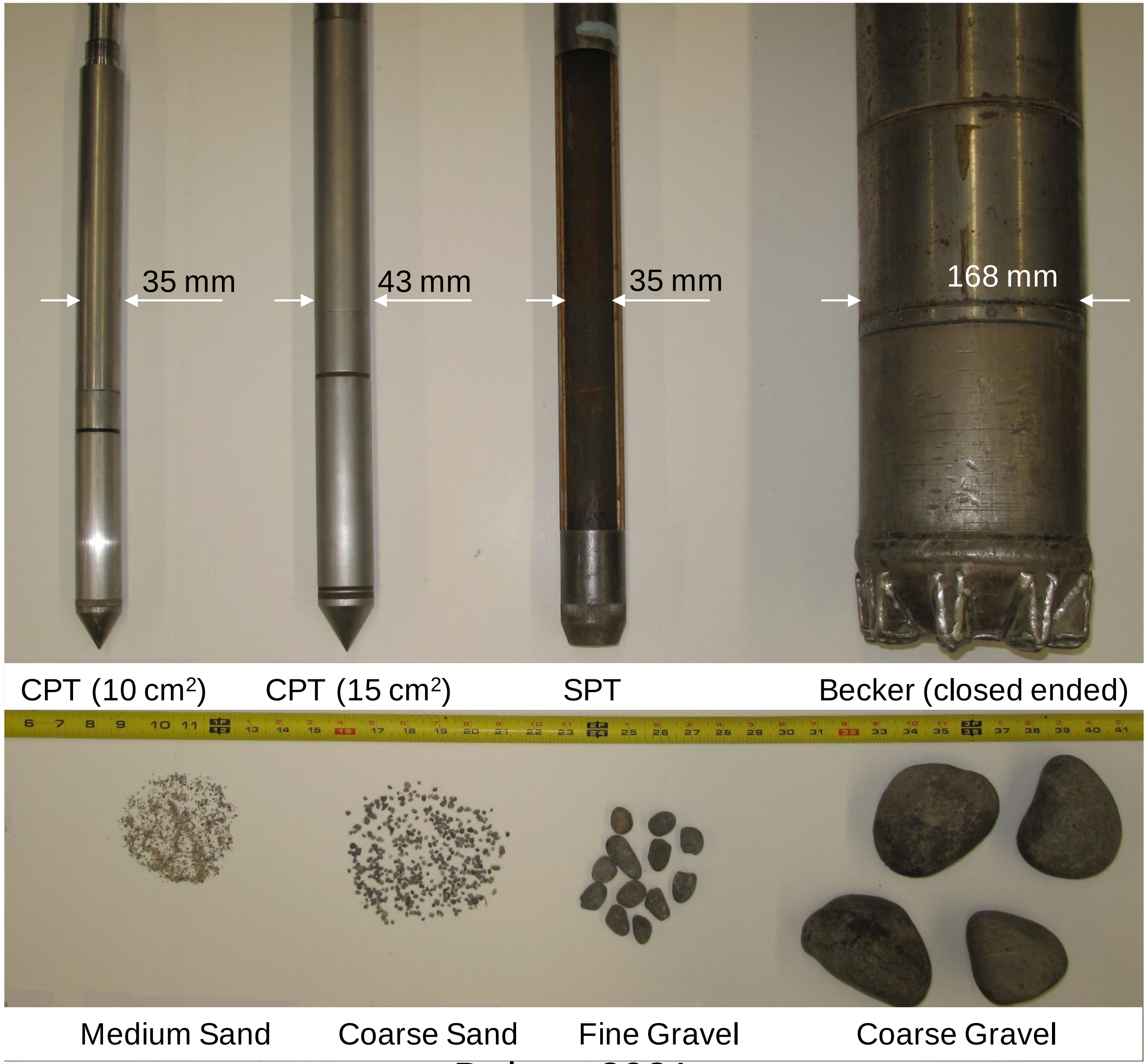


(Jana 2021, Dejong 2021, Athanasopoulos-Zekkos 2022)

Particle size to Penetrometer Scaling

- **The characterization of coarse-grained gravelly soils is difficult** (Dejong 2021).
  - Sampling and laboratory testing difficult due to particle size, disturbance, and sample reconstitution issues.
  - In-situ characterization difficult due to particle-to-probe size effects.

Dejong 2021



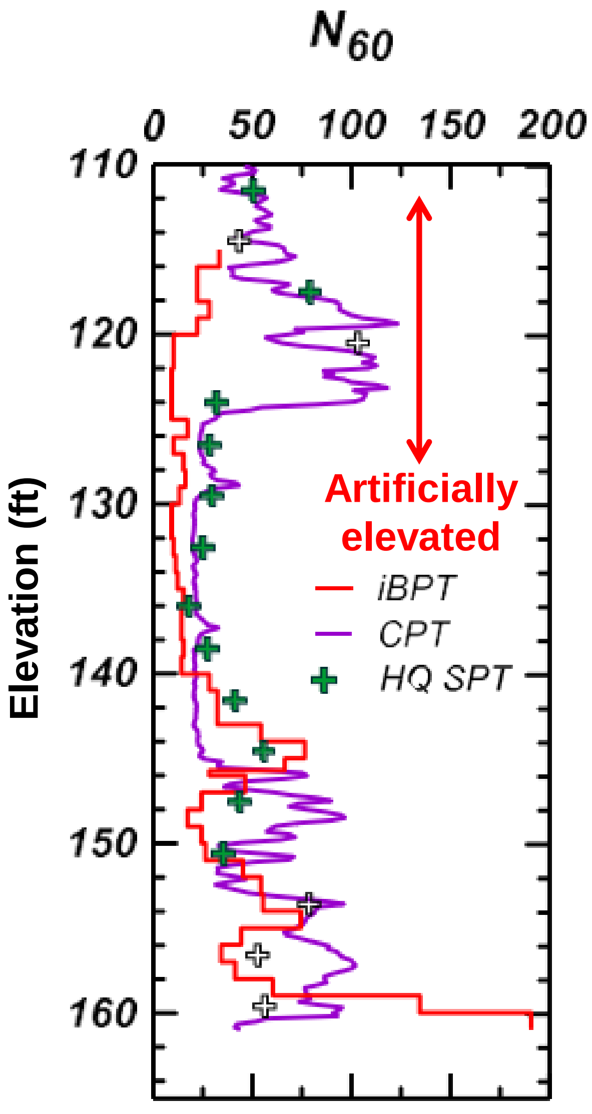
Dejong 2021

In-situ Strength Estimation of Gravelly Soils

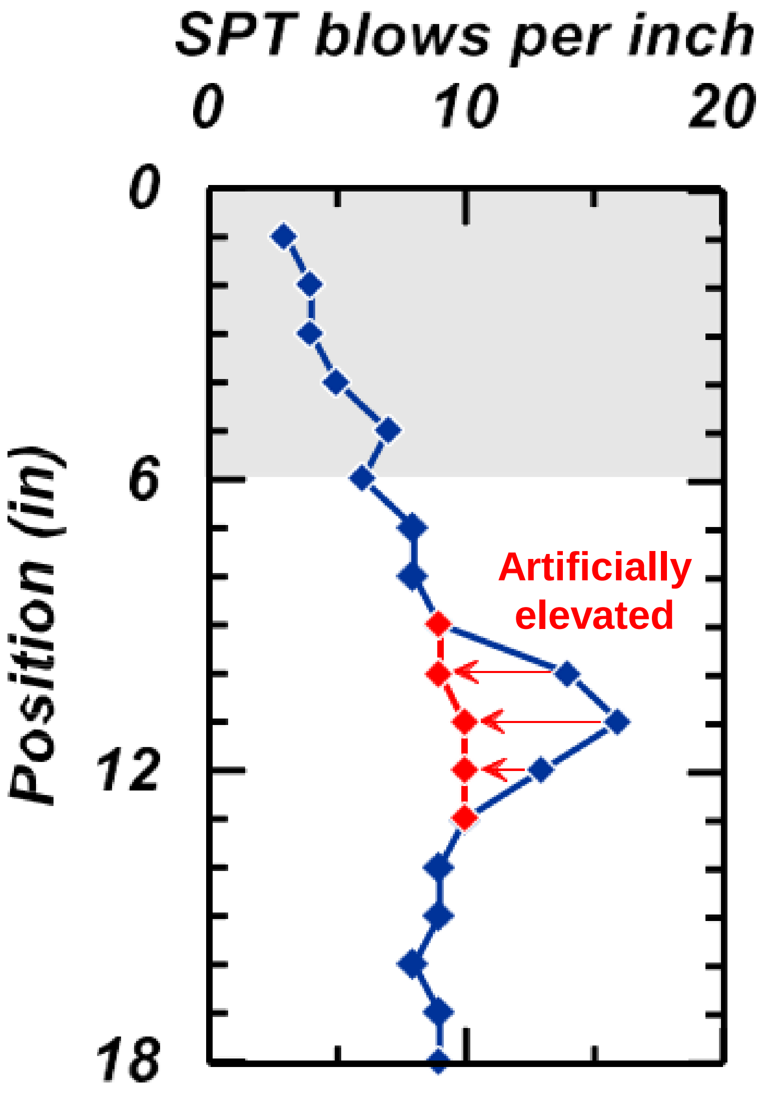
- Conventional in-situ penetration tests **do not perform reliably**.
- The presence of **large particles** can **compromise** the penetration mechanism of the SPT and CPT, resulting in **artificially elevated** measures of penetration resistance.
- Percent gravel, maximum particle size, GSD, and particle hardness all influence SPT N value.
- **Estimated strength parameters for foundation design will be overestimated.**

Kulhawy and Chen (2007)

$$\phi' = 27.5 + 9.2\, log[(N_1)_{60}]$$



Stone Canyon Dam
Dejong 2021



Ghafghazi et al. (2017)

In-situ Strength Estimation of Gravelly Soils

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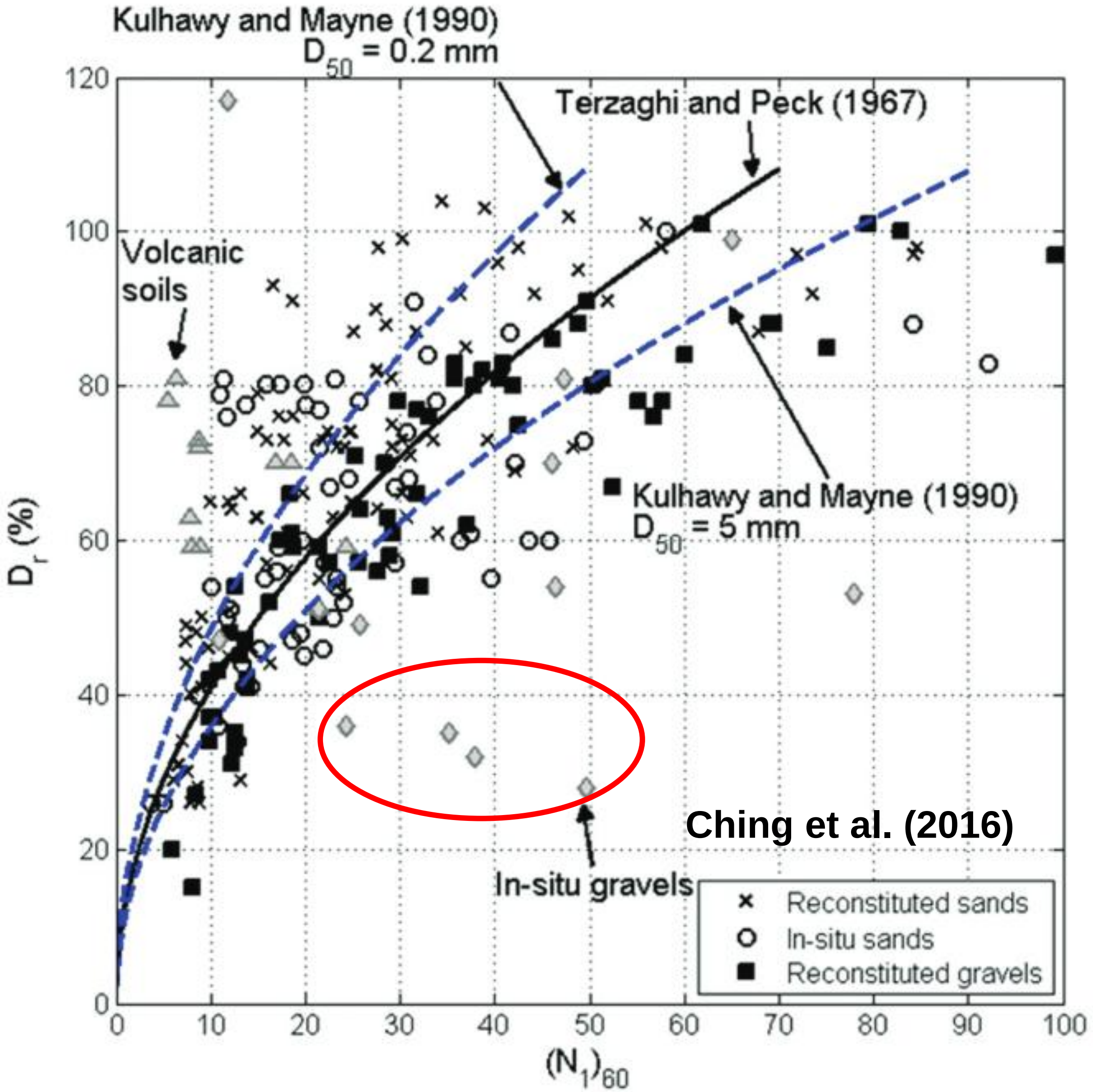
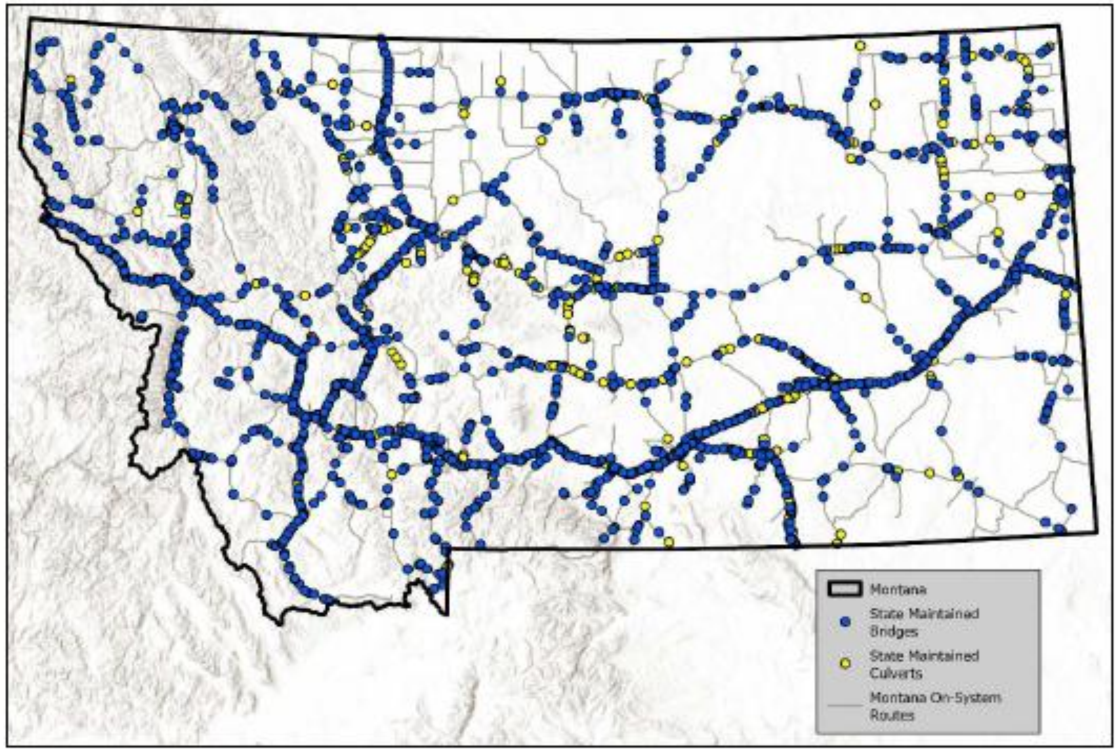
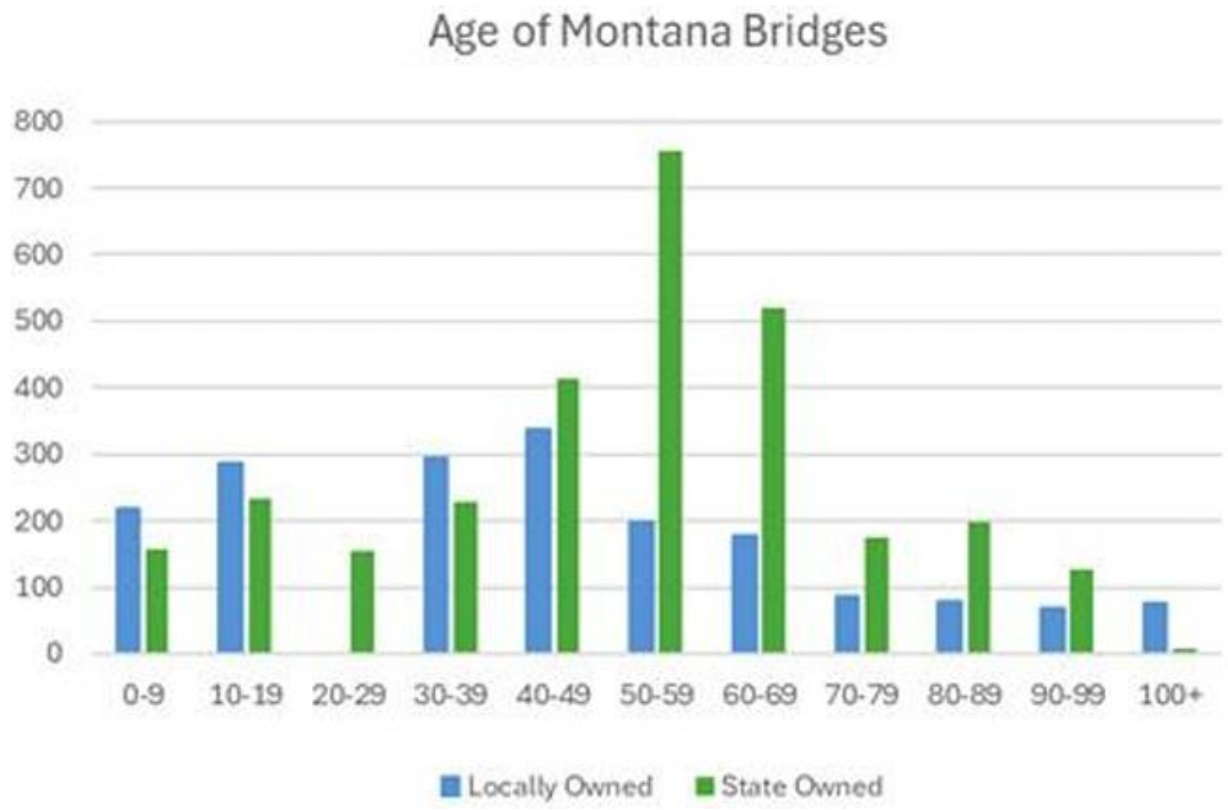


Why do we care?

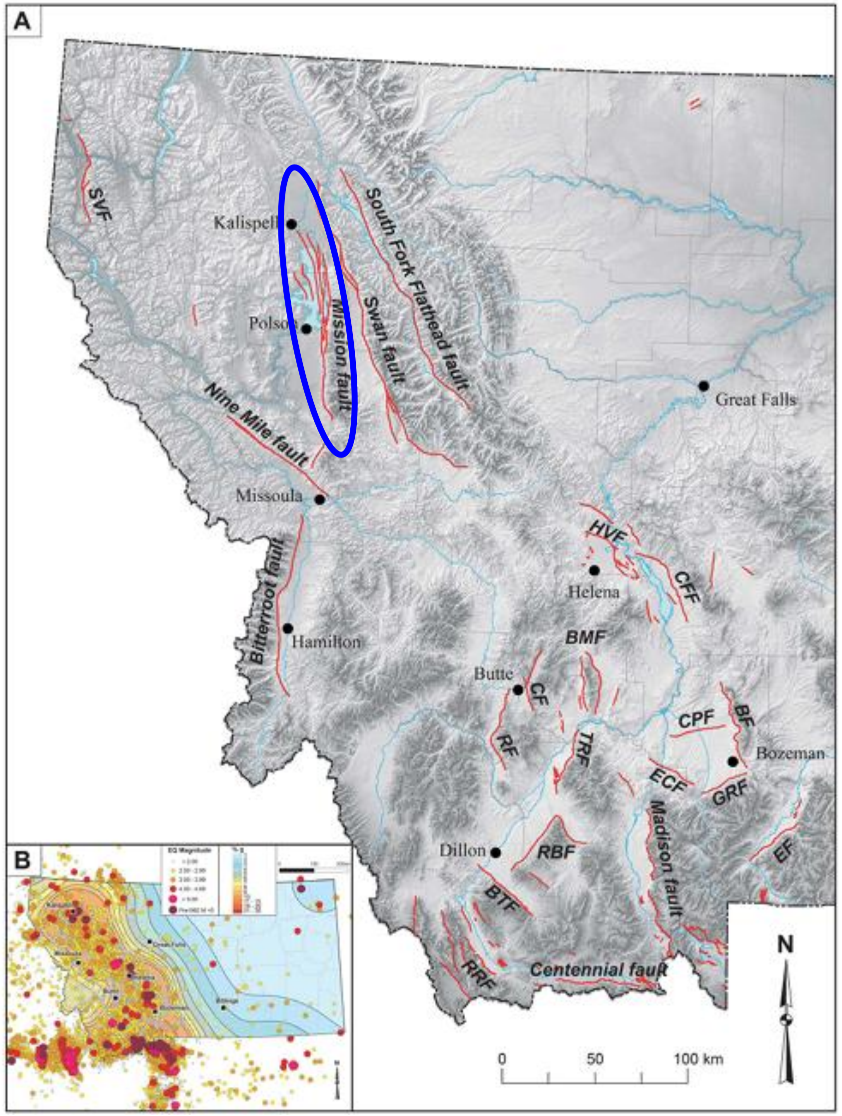
Aging Infrastructure

- *Montana has 5,200 bridges and culverts maintained by the Montana Department of Transportation (MDT).*
- *On average, state-owned bridges are 50 years old with locally owned bridges averaging 45 years old.*
- *As old bridges are repaired and new bridges are being built, it is important to consider challenges with Gravelly soils which are often used as bearing layer for foundation design.*



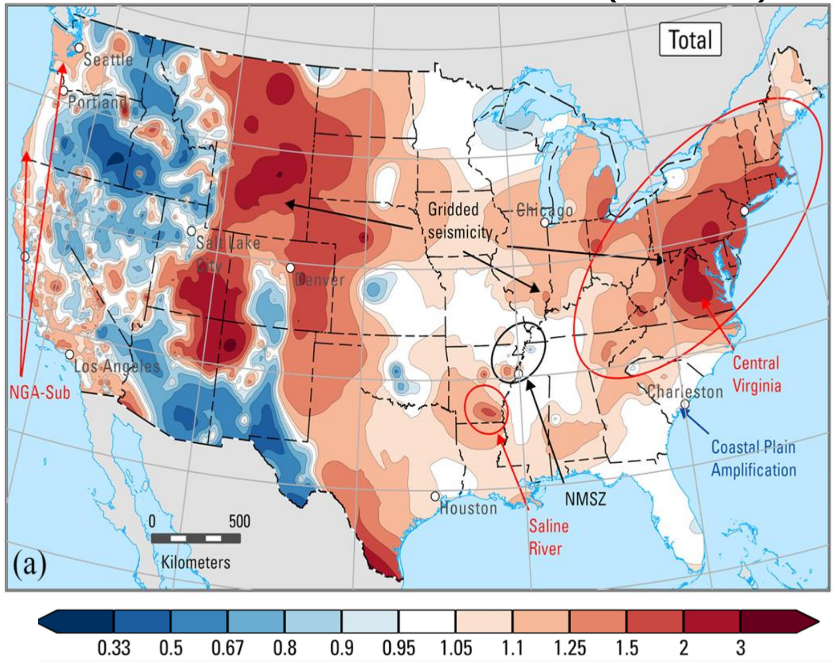


Seismicity In Montana



Major Quaternary fault database for western Montana (MBBG)

***Only **Mission Fault** is Included in National Seismic Hazard Model (NSHM)

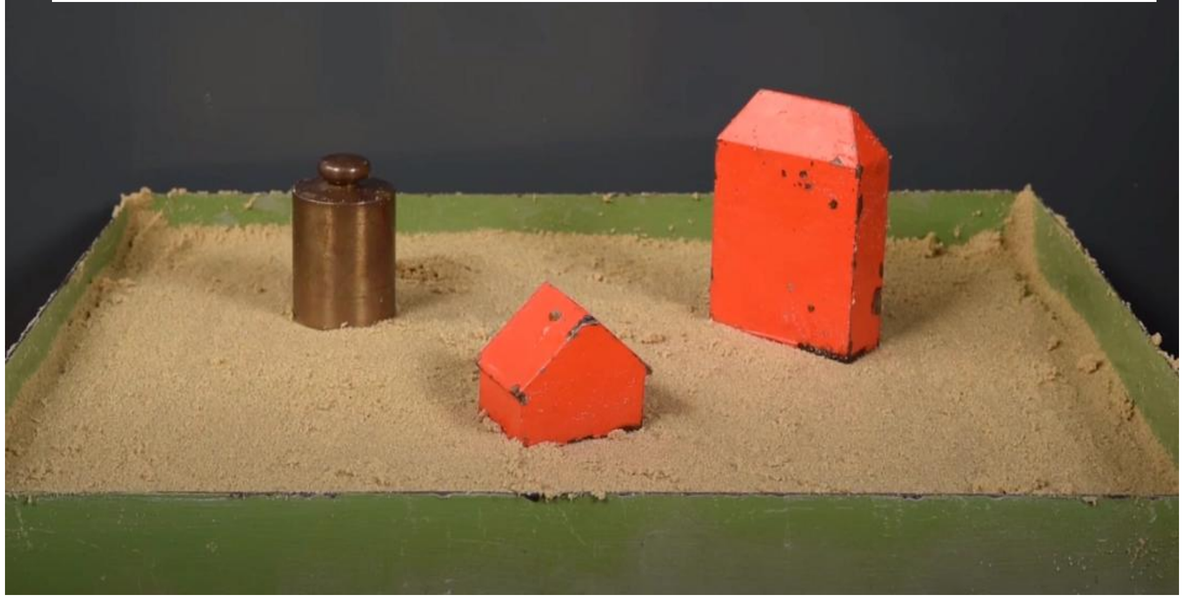


Ratio (g) new model by previous model (USGS 2024)

Can Gravelly Soil Liquefy During an Earthquake?

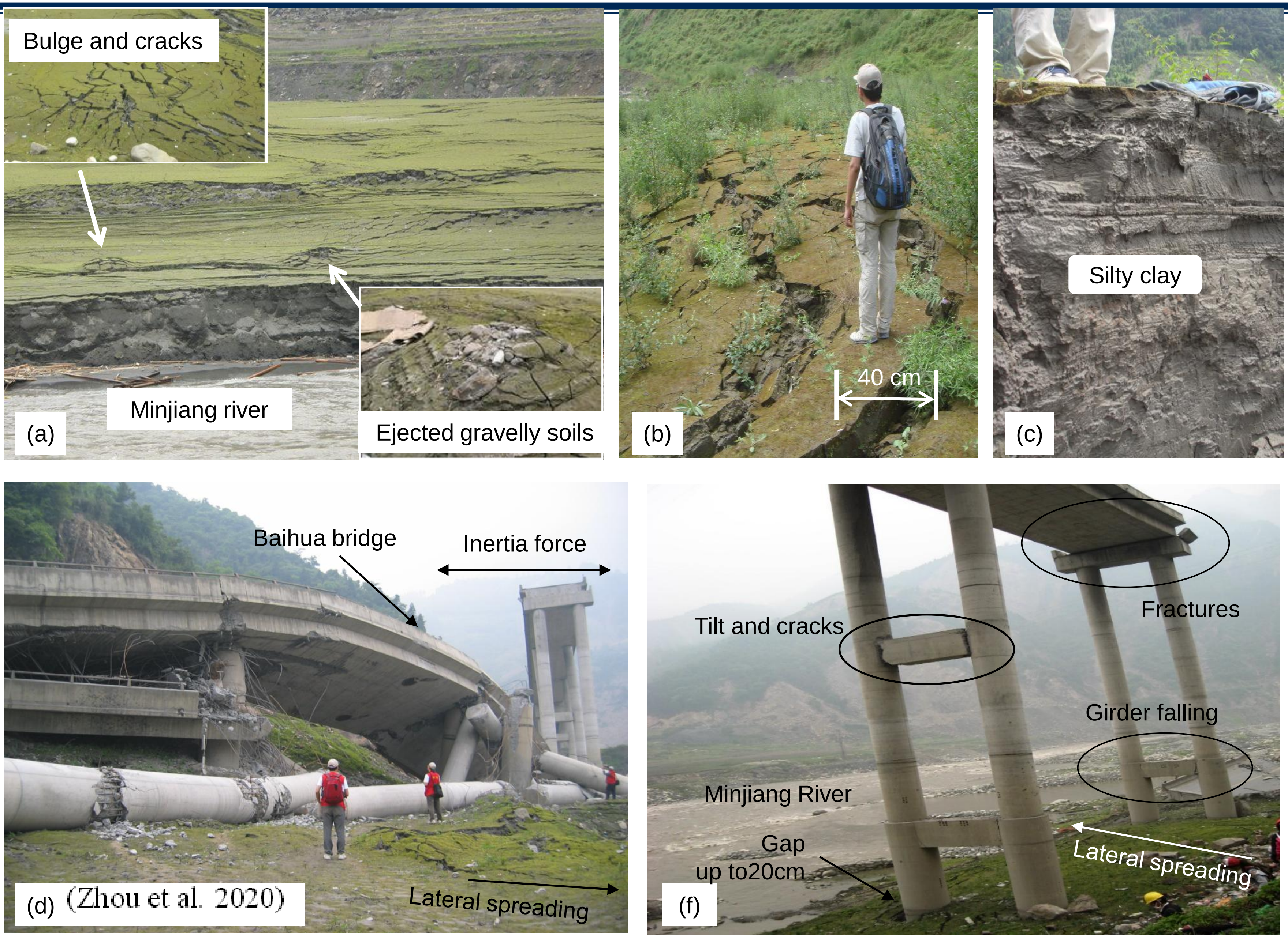
???

Liquefaction: Loss of strength and stiffness of Soil



RWTH Aachen University

Gravel Liquefaction and Its Effects on Infrastructure



Gravel Liquefaction and Its Effects on Infrastructure

M_w = 6.1 2014 Cephalonia EQ (Nikolaou et al. GEER 2014)

2016 Kaikoura EQ (Nikolaou et al. GEER (2014)

M_w = 7.8

M_w = 7.9, 2008 Wenchuan EQ (Zhou et al. 2020)

M_w = 7.8, 2016 Muisne EQ (Consultola 2016)

Gravel Liquefaction and Its Effects on Infrastructure

Case histories involving liquefaction of gravelly soil (Rollins et al. 2022)

Earthquake	Year	M_w	Reference
Mino-Owari, Japan	1891	7.9	Tokimatsu and Yoshimi (1983)
San Francisco, California	1906	8.2	Youd and Hoose (1978)
Messina, Italy	1908	7.1	Baratta (1910)
Fukui, Japan	1948	7.3	Ishihara (1985)
Alaska, USA	1964	9.2	Coulter and Migliaccio (1966), and McCulloch and Bonilla (1970)
Haicheng, China	1975	7.3	Wang (1984)
Tangshan, China	1976	7.8	Wang (1984)
Friuli, Italy	1976	6.4	Sirovich (1996a, b), and Rollins et al. (2020)
Miyagiken-Oki, Japan	1978	7.4	Tokimatsu and Yoshimi (1983)
Montenegro	1979	6.9	Kociu (2004)
Borah Peak, Idaho, USA	1985	6.9	Youd et al. (1985), Andrus (1994), and Harder and Seed (1986)
Armenia	1988	6.8	Yegian et al. (1994)
Limon, Costa Rica	1991	7.7	Franke and Rollins (2017)
Roermond, Netherlands	1992	5.8	Maurenbrecher et al. (1995)
Hokkaido, Japan	1993	7.8	Kokusho et al. (1995)
Kobe, Japan	1995	7.2	Kokusho and Yoshida (1997)
Chi-Chi, Taiwan	1999	7.8	Chu et al. (2000), and Lin et al. (2004)
Kocaeli, Turley	1999	7.6	Bardet et al. (2000)
Wenchuan, China	2008	7.9	Cao et al. (2011, 2013)
Tohoku, Japan	2010	9.0	Tatsuoka et al. (2017)
Cephalonia Island, Greece	2012	6.1	Nikolaou et al. (2014), and Athanasopoulos-Zekkos et al. (2019, 2021)
Iquique, Chile	2014	8.2	Rollins et al. (2014), and Morales et al. (2020)
Muisne, Ecuador	2016	7.8	Lopez et al. (2018)
Kaikoura, New Zealand	2016	7.8	Cubrinovsky et al. (2017)
Durres, Albania	2019	6.4	Pavlides et al. (2020)
Petrinja, Croatia	2020	6.4	Amoroso et al. (2021)

M_w = 5.8 to 9.2

We cannot rule out the possibility of liquefaction of Gravelly soil

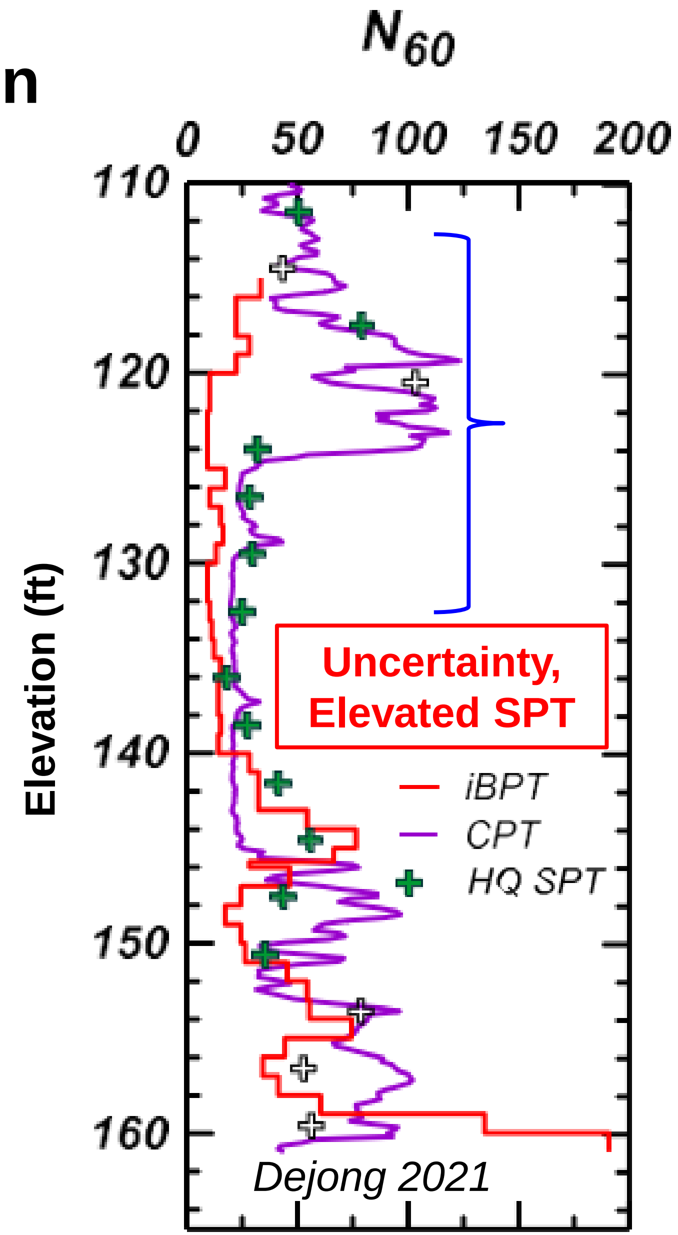
MDT Geotechnical Manual: Liquefaction Evaluation

19.4.2 Liquefaction Potential

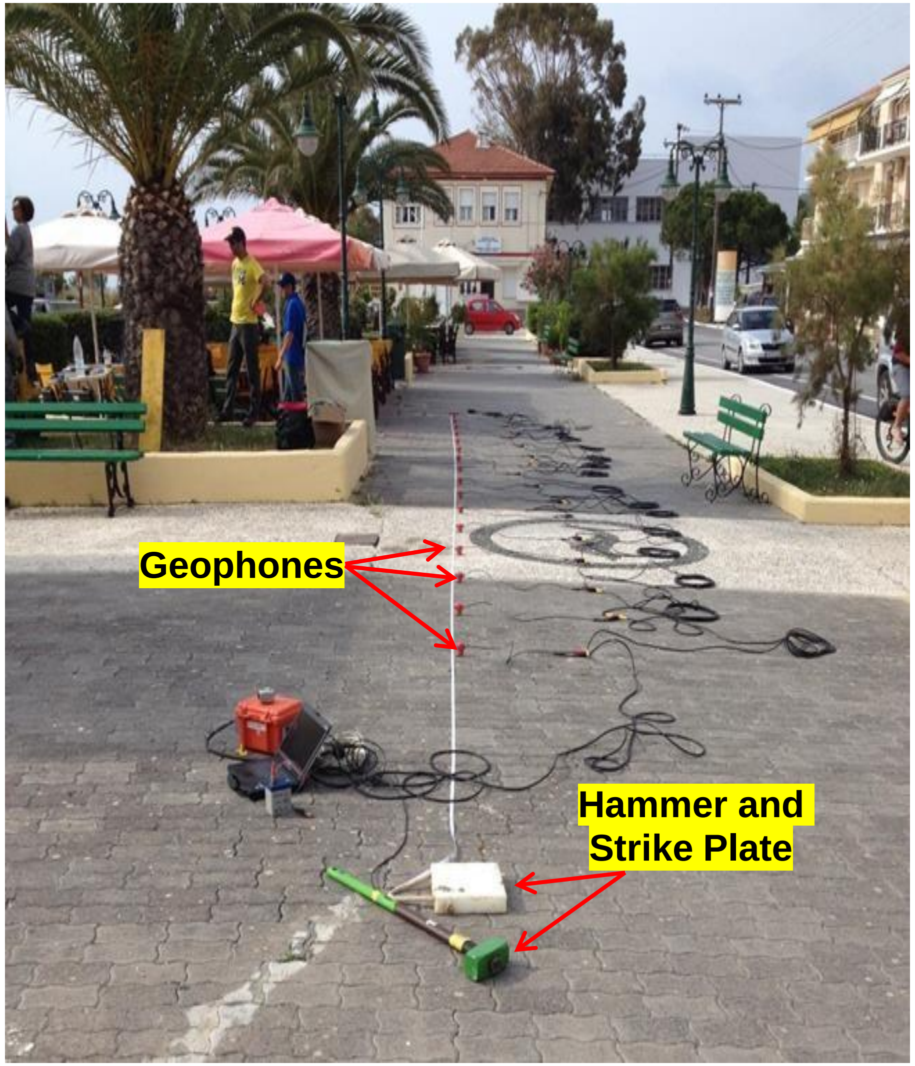
Liquefaction occurs when loose, cohesionless soils located below the groundwater table undergo strong vibratory loading. Porewater pressures within the soil increase as the loose material tends to densify. Soil liquefaction occurs when the increase in porewater pressure equals the effective stress in the soil. In this state, the soil loses shearing strength, potentially leading to bearing failures or slope instability. After the earthquake ground shaking stops, the excess pore pressures dissipate, resulting in settlement. The settlement can effect roadways, retaining walls, bridges, culverts, spread footings and potentially cause downdrag on piles located in the settling soil.

Liquefaction analysis usually begins with a preliminary screening that evaluates three factors to rule out liquefaction. A detailed evaluation of liquefaction potential is not required if one or more of the following conditions occurs at the site:

- The estimated maximum groundwater level at the site is determined to be deeper than 75 ft (25 m) below the existing ground surface or proposed finished grade, whichever is deeper.
- The subsurface profile is characterized as having a minimum SPT resistance, corrected for overburden depth and hammer energy $(N_1)_{60}$, of 30 blows/ft (30 blows/0.3 m), or a cone tip resistance q_c of more than 160 tsf (15 MPa), or if the bedrock is present to the ground surface.
- The soil is clayey, as defined by the recommendations given by Idriss and Boulanger (2006) or Bray and Sancio (2006).

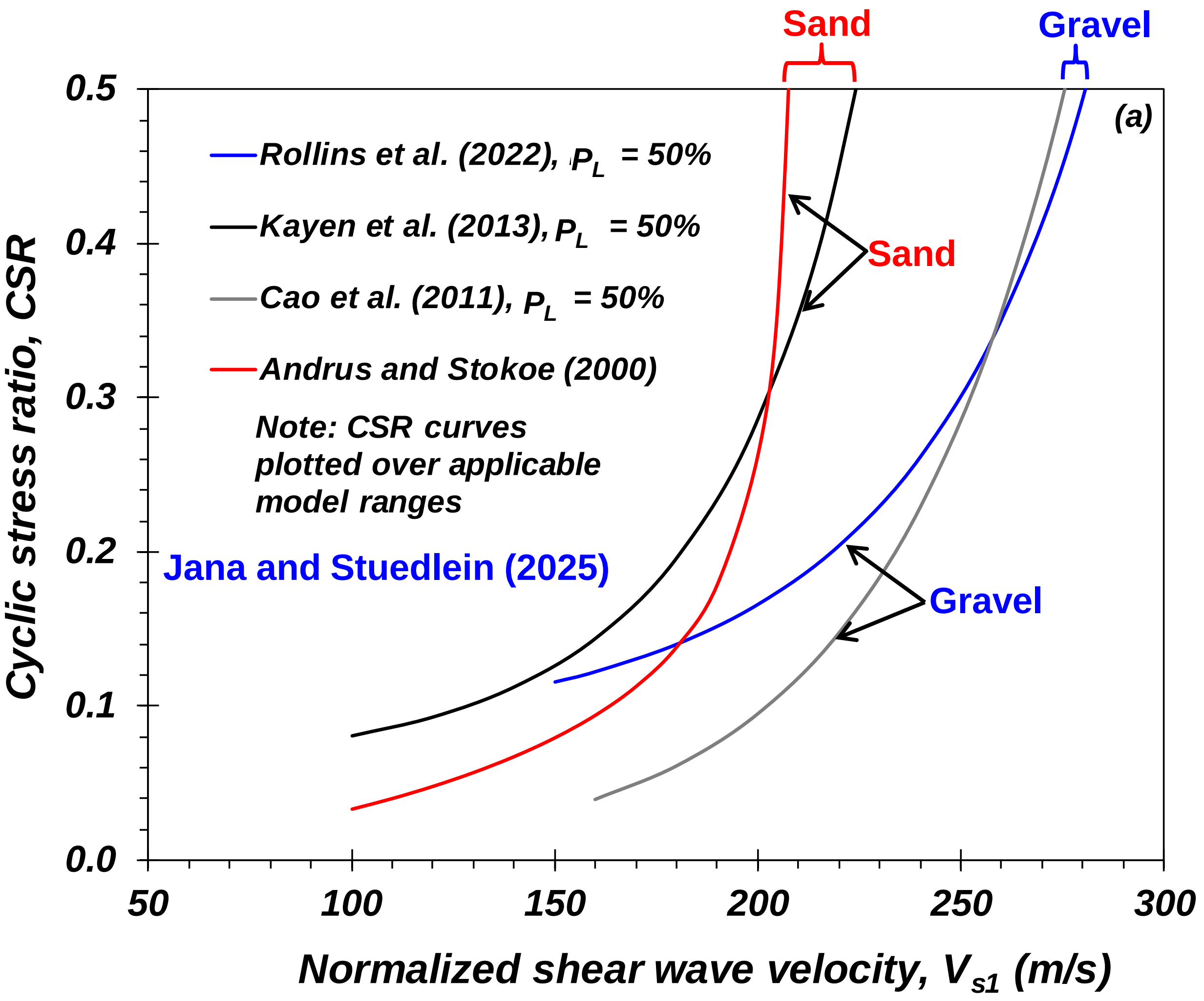


Difference in Cyclic Strength between Gravels and Sand

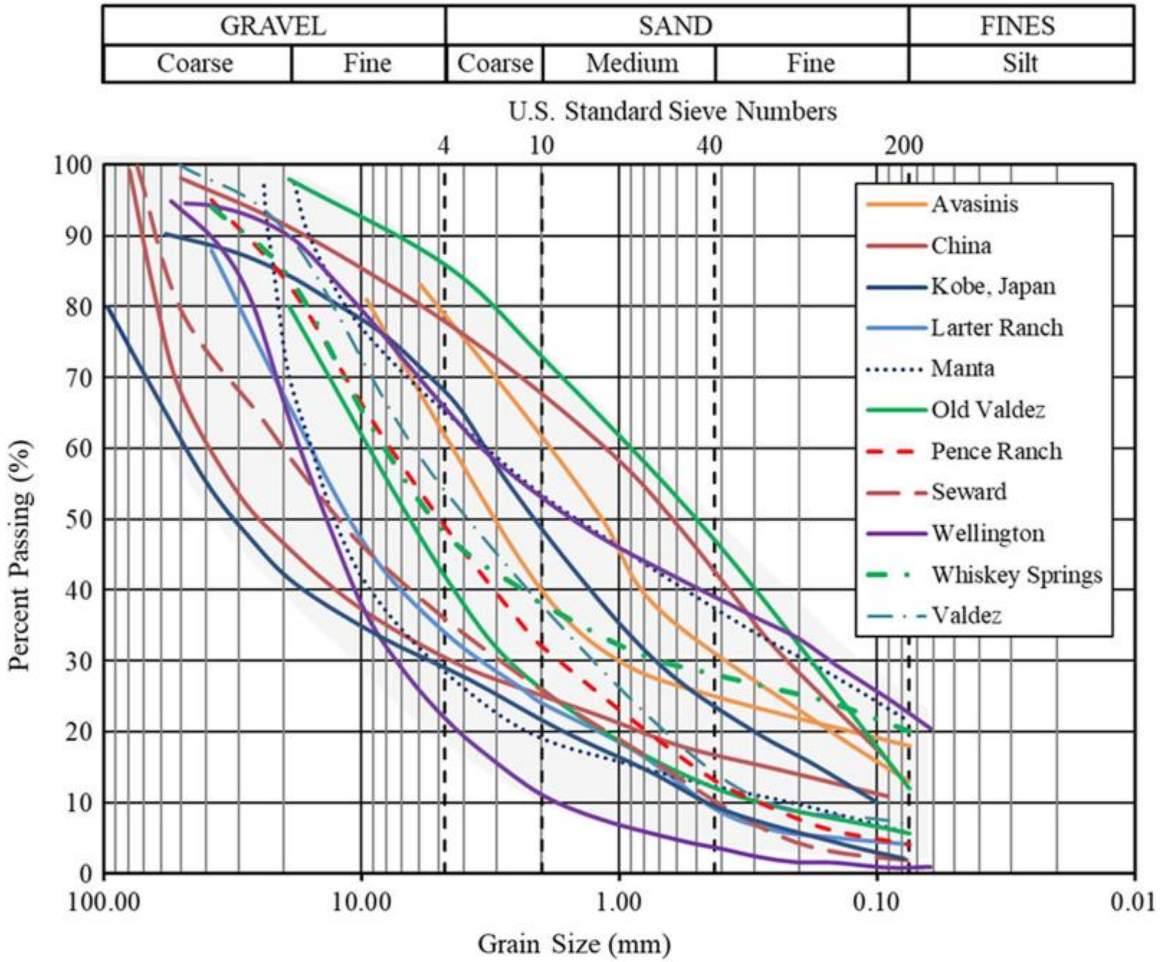


MASW Shear Wave Velocity, V_s Measurements (Hubler 2017)

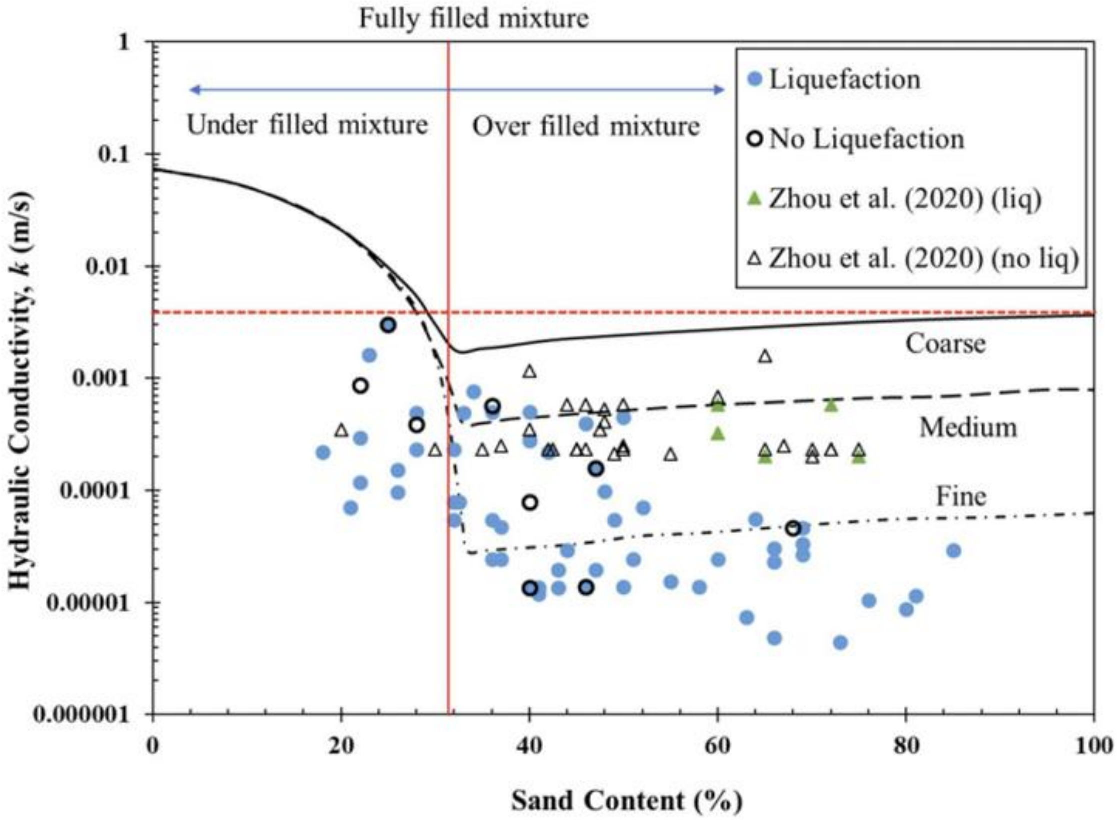
Based on the updated literature, **Gravelly soils has lower cyclic resistance** than **Sandy soils** for the same normalized shear wave velocity.

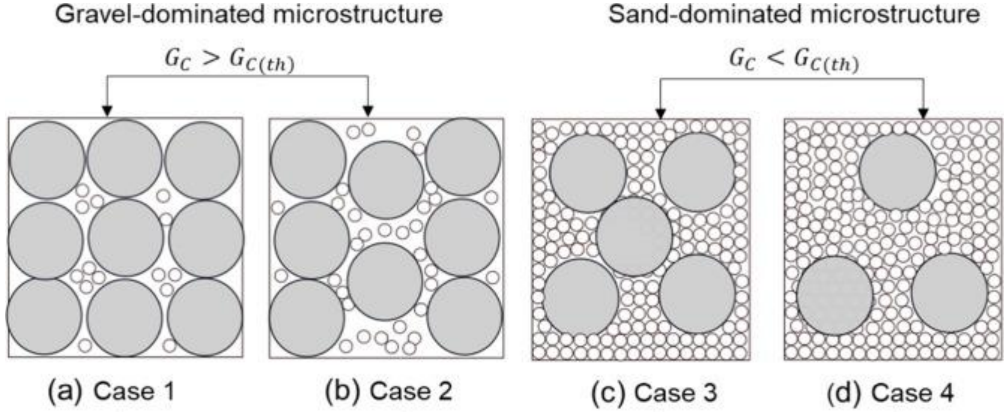


Permeability of Gravelly soils Controls Cyclic Strength

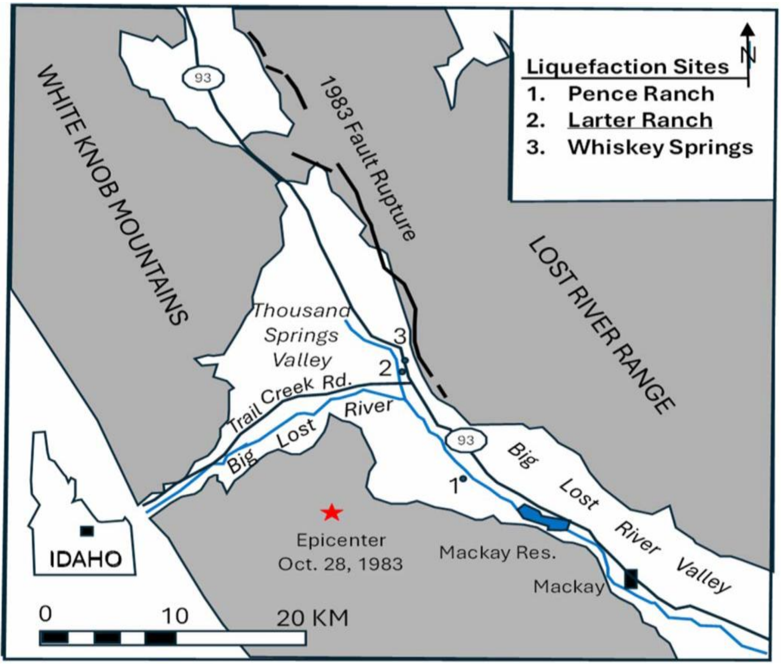


***Liquefaction* case history sites (Rollins et al. 2022)**



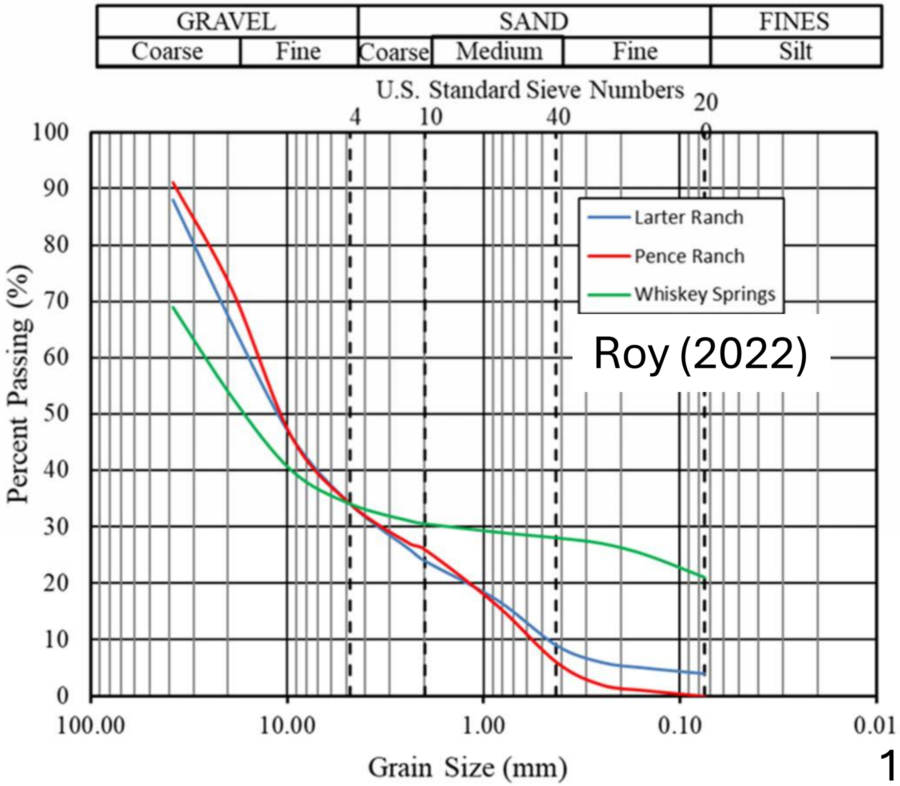


Example Case Study: Borah Peak Earthquake 1983, Idaho, Magnitude M_w= 6.9



Roy (2022)

(USGS)



Roy (2022)

Site Name	No of DPT	Cu	PGA	GC (%)	SC (%)
Pence ranch	4	22	0.44	34	66
Larter Ranch	4	37.3	0.44	45	55
Whiskey spring	2	47	0.52	64	36

Methodology (Jana and Stuedlein 2025)

Determination of critical shear strain to initiate liquefaction

Rollins et al. (2022; REA22) developed a CRR model using 174 case histories of gravelly soils

$$CRR = \exp\left(\frac{3.88 \cdot 10^{-7} \cdot V_{s1}^{3} - 1.6 \cdot M_w - \ln\left(\frac{1-P_L}{P_L}\right)}{4.95}\right)$$

$$\gamma_{cl} = exp\left(\frac{3.88 \cdot 10^{-7} \cdot V_{s1}^{3} - 1.6 \cdot M_w - ln\left(\frac{1-P_L}{P_L}\right)}{4.95}\right) \cdot \left[\frac{\sqrt{\sigma'_{v0}}}{\frac{0.1 \cdot \rho \cdot {V_{s1}}^{2}}{1 + \left\{\frac{\gamma_{cl}}{0.0046 \cdot (C_u)^{-0.197} \cdot (\sigma'_0)^{0.52}}\right\}^{0.84}}}\right]$$

$$\gamma_c = 0.65 \cdot \left(\frac{a_{max}}{g}\right) \cdot \frac{\sigma_{v0} \cdot r_d}{G_{max} \cdot \left(\frac{G_c}{G_{max}}\right)}$$

$$\gamma_{max} = \left(\frac{a_{max}}{g}\right) \cdot \frac{\sigma_{v0} \cdot r_d}{G_{max} \cdot \left(\frac{G_c}{G_{max}}\right)}$$

$$r_u = \frac{p \cdot f \cdot N_{eq} \cdot F \cdot (\gamma_c - \gamma_{tp})^{s}}{1 + f \cdot N_{eq} \cdot F \cdot (\gamma_c - \gamma_{tp})^{s}}$$

CRR- Cyclic Resistance Ratio

V_{s1} - over burden corrected

Shear wave velocity

γ_{cl}- critical shear strain

P_L- probability of liquefaction

σ'_{v0}- Effective stress

M_w - Magnitude of earthquake

C_u - Coefficient of uniformity of soil

P, f, s = Fitting parameters

f = Dimensionality of loading

N_{eq} = Number of equivalent cycle

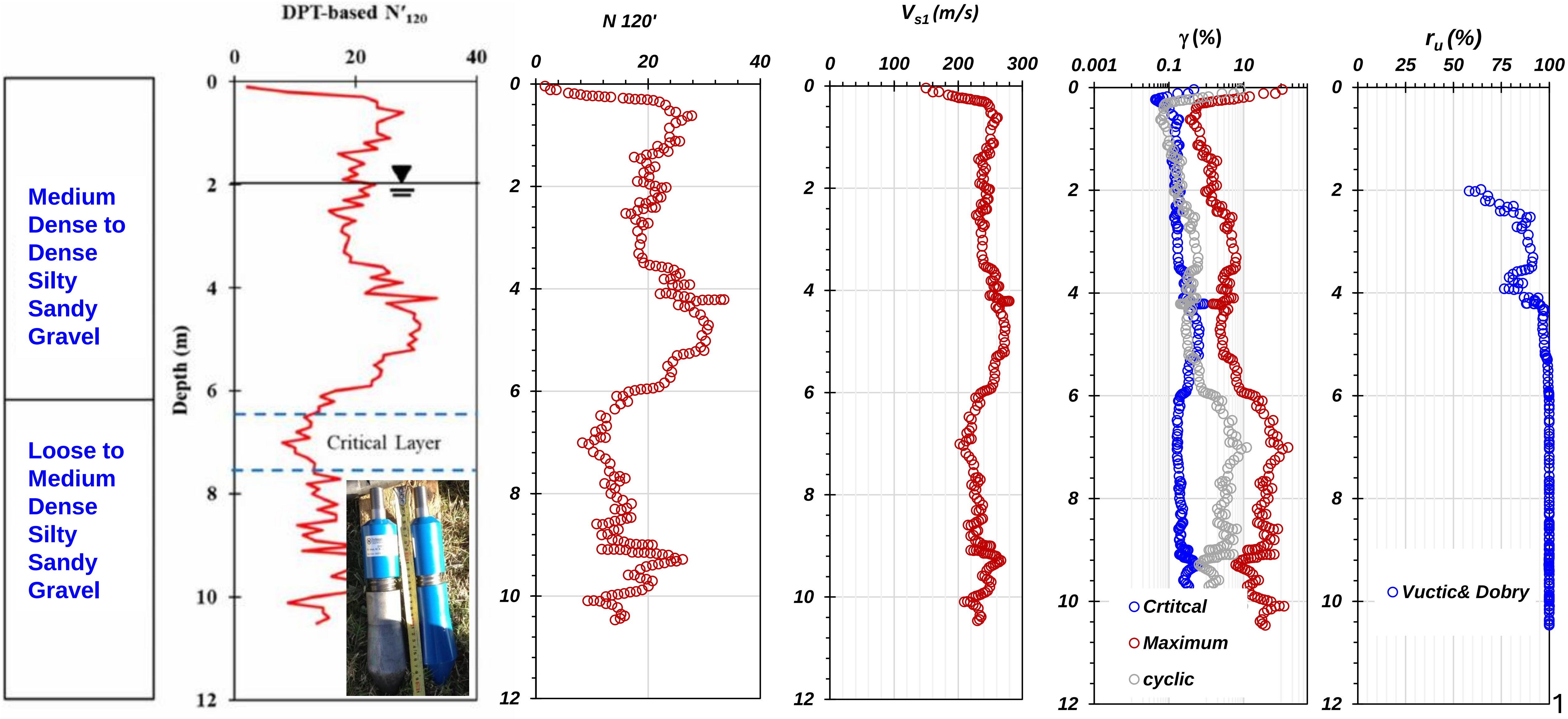
γ_{tp} = Threshold shear strain

F = Controls the rate of r_u

FC = Fineness content

Borah Peak Earthquake 1983, Idaho

Site: Larter Ranch, Idaho, M_w= 6.9, PGA = 0.44



Concluding Remarks

- *Conventional in-situ penetration tests* ***do not perform reliably*** *for Gravelly soils.*
- *If using SPT, suggest using* ***N per inch*** *reading.*
- ***Shear wave*** *velocity could provide insight of static and dynamic response of Gravelly Soils.*
- *Suggest using* ***Dynamic Penetration Test (DPT)*** *for Gravelly soils.*
- *Our newly developed method could provide simplified system response of deposit.*

DPT: Less Expensive

BPT: Very Expensive

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Questions?

NSF

Extra Slides

Standard Penetration Test (SPT)

Important Notes

- Advantages:
  - Relatively quick Test, simple, inexpensive, widely used
  - Provides a sample!!
  - Can use in dense materials
- Disadvantages
  - Result are affected by many variables
  - Inaccurate in **Gravels** (Do not use "N")
  - For Gravels use ITBP (Instrumented Becker Penetration Test)

Large Gravel Particles cannot be sampled using SPT and N value will be affected by Gravel.
If SPT is the only option measure N value for every inch penetration.