Abstract Geotechnical Seismic Isolation (GSI) can be defned as a new category of seismic isolation techniques that involve the dynamic interaction between the structural system and geo-materials. Whilst the mechanism of various GSI systems and their performance have already been demonstrated through diferent research methods, there is a missing link between fundamental research and engineering practice. This paper aims to initiate the development in this direction. A new suite of equivalent-linear foundation stifness and damping models under the same framework is proposed for four GSI confgurations, one of which is a novel combination of two existing ones. The exact solutions for the equivalent dynamic properties of fexible-base systems have also been derived that explicitly include the foundation inertia and the strain-dependent equivalent damping of foundation materials, which are both signifcant for GSI systems. The application of the proposed analytical design models has been illustrated through response history analyses and a detailed handcalculation design procedure has also been outlined and demonstrated. Keywords Geotechnical seismic isolation · Soil–structure interaction · Equivalent-linear · Flexible-base period · Foundation damping · Design procedure 1 Introduction Resilience is one of the core structural design objectives. Amongst the various low-damage resilient seismic design approaches, seismic isolation is a well-accepted and efective strategy. Conventional seismic isolation technologies based on the use of discrete bearings are well-established and increasingly popular all around the world. However, the additional cost of construction is relatively high, and it requires specialist design skills that are not conveniently accessible. Whilst there have been some attempts to develop low-cost base isolation bearings (Kelly 2002; De la Llera et al. 2004), applications of the conventional seismic isolation technologies are limited to large-scale, more expensive or important structures such as government and commercial buildings, as well as luxurious residential blocks. \* Hing-Ho Tsang htsang@swin.edu.au 1 Swinburne University of Technology, Melbourne, Australia 3882 Bulletin of Earthquake Engineering (2023) 21:3881–3904 1 3 Global sustainable development as an overarching goal requires the reduction of inequality and poverty within and amongst countries. Universal earthquake safety and resilience are indispensable for achieving this common goal. It is essential to develop earthquake protection approaches that are suitable for all, in terms of both afordability and accessibility. Apart from developing a lower-class version of a successful advanced technology, it is desirable to rethink the needs and capabilities of the larger community, adapt to the common structural confguration and routine construction practice, and ensure sustainable consumption of precious raw materials. A fundamentally diferent earthquake protection approach is needed. Geotechnical Seismic Isolation (GSI), a term coined by Tsang (2009), is an emerging earthquake protection approach that can potentially address the challenges and achieve the goals discussed above. GSI can be defned as a new category of seismic isolation techniques that involve the dynamic interaction between the structural system and geo-materials, natural or modifed. As the design of GSI systems focuses on the modifcation of the foundation soils, no changes to the structural confguration is required. Most GSI techniques are based on the replacement of original soils, involving excavation and backfll, which are standard site formation techniques that do not require specialist skills. Hence, the additional design and construction cost would be minimal. If waste materials are upcycled to create the modifed foundation soil for GSI, this can further reduce the cost and at the same time contribute to sustainable consumption as part of a circular economy (Tsang 2012; Hernández et al. 2020). Given the low replacement rates of buildings and infrastructure, especially in the developed parts of the world, seismic retroftting techniques are of paramount importance to achieve the resilience of the whole community. An ideal seismic retroftting technique has to be cost-efective, less invasive, and compatible with the original structural confguration and architectural features. Gatto et al. (2021) have recently proposed a GSI system based on the injection of expandable polyurethane into the ground through small holes across the footing slab. Somma et al. (2022) have proposed to remove the soils around the peripheral of the footing to create lateral disconnection for realising the mechanism of GSI. Both innovative techniques have been shown to be efective seismic retroftting strategies. These techniques can also be used for new constructions to minimise the need for excavation. GSI can be achieved by introducing a low-modulus layer and/or a sliding interface surrounding the foundation for decoupling the structure from the ground shaking. Low-modulus materials are used to reduce the horizontal and rocking stifnesses of the foundation system for creating a GSI system that exploits the benefcial efects of dynamic soil–structure interaction (Tsang 2008), whilst geosynthetic liners (Yegian and Kadakal 2004; Yegian and Catan 2004), sand (Tsiavos et al. 2020) or stone pebble (Banović et al. 2019) can be used to create the low-friction sliding interface. These two major GSI mechanisms are analogous to the conventional structural seismic isolation systems based on the use of laminated rubber bearings and spherical sliding bearings respectively (Tsang 2009). This paper focuses on the former isolation mechanism. Various confgurations of GSI systems and the choices of low-modulus materials will be discussed in Sect. 2. With the lower horizontal and rocking stifnesses of the foundation system, the seismic demand can generally be reduced due to the increase in the natural period of the whole soil-foundation-structure system. Meanwhile, the seismic displacement demand would be concentrated at the fexible foundation layer, such that the deformation of the superstructure can be reduced (Tsang and Pitilakis 2019). The mechanism is fundamentally diferent from the vibration screening or scattering technique that is based on wave refection, difraction and damping (Woods 1968; Beskos et al. 1986; Alzawi and El Naggar 2011) Bulletin of Earthquake Engineering (2023) 21:3881–3904 3883 1 3 or those techniques based on the use of meta-materials that were explored in more recent times (Cheng et al. 2020), even though they can also be classifed as GSI techniques by defnition. In the past one and a half decades, numerical simulations and analytical modelling have been conducted by various researchers to demonstrate the efectiveness of GSI systems (Tsang et  al. 2009, 2012a; Pitilakis et  al. 2015; Anbazhagan et  al. 2015; Abdullah and Hazarika 2016; Brunet et al. 2016; Dhanya et al. 2020; Forcellini and Alzabeebee 2022). Experimental testing and feld measurement have also been performed to confrm the isolation mechanism and to evaluate their performance (Kaneko et al. 2013; Nikitas et al. 2014; Nappa et al. 2016; Tsiavos et al. 2019; Tsang et al. 2021; Pitilakis et al. 2021). It comes to the stage when appropriate design models, procedures and guidelines are needed for systematic design in real applications (Tsang and Pitilakis 2019). This study aims to initiate the development in this direction. 2 GSI system confgurations What is ideal seismic isolation? It would be ideal if the whole structure can be completely decoupled from the ground shaking (C0) (refer Fig. 1a). Advanced technologies like magnetic levitation or suspension can possibly be adopted to achieve this. However, it is considered by many as something impractical for the civil engineering industry. Having said that, the magnetic levitation (maglev) train was once considered an insane concept. It took decades for the technology to move out of the academic realm into reality. Cost is undoubtedly a key issue and there are two dimensions here: construction cost and operational cost. If an advanced technology is introduced to an ordinary low-tomedium-rise residential building, even the use of laminated rubber bearings is considered too expensive. If the same technology is used for isolating a small piece of asset that is worth billions of dollars/euro/pounds, the asset owner may be keen on exploring such an ideal option as the cost incurred is only a tiny fraction of the value of the asset itself. On the other hand, if the electromagnetic system is required to run non-stop, the operational cost would be excessive. However, if the system is applied in tandem with an existing earthquake early warning (EEW) system (Chan et al. 2019), in that the maglev mechanism is only activated when an earthquake occurs, the operational cost could be dramatically reduced. 2.1 GSI material around foundation With the ideal isolation strategy in mind, it is sensible and logical to design a seismic isolation system that is based on flling the “levitated” gap with low-modulus materials (C1) (refer Fig. 1b). This is also the basic confguration of a GSI system as frstly proposed in Tsang (2008). It is well known that the hyperloop with an airless tube exhibits superior performance to the existing maglev train system. Likewise, materials with lower modulus would likely lead to better seismic isolation efects, provided that the structure being supported can fulfl a range of serviceability requirements. Considering the compatibility with the natural environment and the availability of materials, mixtures of soil and waste tyre rubber, also known as rubber–soil mixtures (RSM), have been proposed as a desirable choice for GSI (Tsang 2008). The potential of upcycling a huge amount of waste tyre rubber is an additional beneft (Tsang 2012). Supposedly, it 3884 Bulletin of Earthquake Engineering (2023) 21:3881–3904 1 3 Fig. 1 Various confgurations of geotechnical seismic isolation (GSI) systems that are based on reduced foundation stifnesses Bulletin of Earthquake Engineering (2023) 21:3881–3904 3885 1 3 is desirable to consume as much waste tyre rubber as possible in each building project. However, there could be complications when it requires large-scale excavation and lateral support. Hence, a thickness of one to three metres appears to be optimum. Other possible candidates of low-modulus materials include EPS geofoam, bentonite slurry, soil–polyurethane (SPU) mixtures, super absorbent polymer (SAP) and cushions of water or pressurised air. 2.2 Lateral disconnection only In fact, an obvious approach to partially achieve the ideal isolation strategy is to isolate only the peripheral of the embedded shallow foundation from the surrounding soil, but not below it (C2) (refer Fig. 1c). This can reduce both the horizontal and rocking stifnesses of the foundation system. This is known as lateral disconnection as investigated by Somma et al. (2022). An advantage of this technique is that it can be applied to existing structures sitting on a shallow foundation as a seismic retroftting measure, given that there is enough space from surrounding structures. Noted that the gap width was designed as 0.6 m at prototype scale in Somma et al. (2022). As the lateral contact between the shallow foundation and the peripheral soil is completely removed, the radiation damping of both the horizontal and rocking motions of the foundation system would be signifcantly reduced. It would take longer to dissipate the vibrational energy of the structure during and after an earthquake. Hence, a balanced solution is to backfll the gap with a low-modulus high-damping material (C2a) (refer Fig. 1d) like RSM, EPS geofoam, SPU or SAP. Whilst the foundation horizontal and rocking stifnesses can still be reduced, a high level of radiation and material damping can be achieved at the same time. Analytical modelling of this GSI confguration based on EPS geofoam has been conducted by Karatzia and Mylonakis (2017), whilst numerical simulation based on RSM has been conducted by Xu (2009) under the supervision of the author of the current paper. 2.3 GSI material below foundation with no embedment If the lateral disconnection approach as discussed in the previous sub-section is a desirable isolation strategy, it is actually sensible to design the foundation system of new construction with no embedment, if possible, which can remove the stifness restraints from the peripheral soil. Meanwhile, it would be desirable to also incorporate a base isolation layer below the foundation such that both the horizontal and rocking stifnesses can be further reduced (C3) (refer Fig. 1e). This confguration has also been explored by Xu (2009) as a scenario when there is a limitation to construct beyond the peripheral of the building foundation. Whilst a base isolation layer is best suited for new constructions, Gatto et al. (2021) have proposed the use of polyurethane injection as a GSI method for existing structures (C3a). Polyurethane is injected into the foundation soil through machine-crafted small holes across the thickness of the footing. The injected polyurethane would then be expanded in the soil to become a GSI system of polyurethane columns. The centre-to-centre distance between injection holes and the desired coverage of GSI columns in terms of the crosssectional area have to be carefully designed based on the expansion coefcient of the polyurethane and the properties of the natural foundation soil. This GSI approach of partially flling the foundation soil is depicted using a simple, idealised diagram in Fig. 1f. 3886 Bulletin of Earthquake Engineering (2023) 21:3881–3904 1 3 2.4 GSI material below foundation with lateral disconnection However, embedment is usually preferred, or even unavoidable, for many structures. Basement levels are designated for some buildings too. For those scenarios, a hybrid approach can be adopted to enhance the isolation efects. For new constructions, a combination of the lateral disconnection technique and a low-modulus layer below the foundation can be designed such that the stifness restraints can be minimised (C4) (refer Fig. 1g). This hybrid GSI approach will be explored for the frst time through response history analysis in Sect. 3.6 of this paper. Furthermore, the hybrid approach can also be explored for retroftting existing structures. Whilst lateral disconnection was proposed primarily for reducing seismic vulnerability of existing structures (Somma et al. 2022), the polyurethane injection technique proposed by Gatto et  al. (2021) appears to be a suitable companion for enhancing the GSI efect (C4a) (refer Fig. 1h). This seismic retroft technique can be regarded as a combination of the two approaches as introduced in Sects. 2.2 and 2.3. 3 Lumped‑parameter analytical design models For the analysis and design of the various GSI systems described in Sect.  2, idealised models can be developed under the same framework to maintain consistency and facilitate direct comparison. Considering a building (or a bridge pier) that is sitting on a shallow foundation (refer Fig. 2a) with a base of arbitrary shape, a lumped-parameter model as shown in Fig.  2b can be used to capture the key dynamic properties of the structure and the foundation soil. The structure can be idealised as an SDOF system, with a lumped mass representing the structural mass, mstr, that is participating in the fundamental mode of vibration, at an efective height of Hef measured from the base of the foundation. Nonlinearity of foundation materials can be taken into account by the shear strain-dependent secant shear modulus, Gsec, the corresponding shear wave velocity, ( VS ) sec, and the Fig. 2 a Case study building sitting on a shallow foundation, and b the corresponding lumped-parameter model for seismic analysis Bulletin of Earthquake Engineering (2023) 21:3881–3904 3887 1 3 equivalent viscous damping, 휁eq, through the use of the equivalent-linear method. A specifc set of equivalent-linear elastic springs and viscous dashpots can be used to represent the stifness and damping properties of the soil-foundation system of each GSI confguration as summarised in Table 1. A detailed description of the full lumped-parameter model for the benchmark confguration (C1) discussed in Sect. 2.1 can be found in Tsang and Pitilakis (2019), based on Hall (1967), Whitman and Richart (1967), Kausel and Roesset (1975), Pais and Kausel (1988), Gazetas (1991), Stewart et al. (2003) and Dobry (2014). The individual coefcients in the stifness and damping models are described in some detail in this section. The shallow foundation is assumed as a rigid plate with mass, mf , and thickness, hf , sitting on a uniform, isotropic, homogeneous viscoelastic half-space. The shape of the foundation base can be of any shape. For the application of the lumped-mass analytical model, the base area has to be circumscribed by a rectangle with dimension 2L × 2B, with L>B, given that L is not signifcantly larger than B. For translational motions in the x-direction, Rx is the radius of an equivalent circle that has an area equal to the base area of the foundation, Ab, i.e. The equivalent radius for rocking motions, R휃, can be computed by equating the second moment of area about the centroidal y-axis, e.g. for a rectangular foundation with dimension 2L × 2B: The mass moment of inertia of the structure and that of the foundation about the centroidal y-axis across the base of the foundation are, respectively: 3.1 Horizontal stifness models (kx) For the benchmark confguration with GSI material around the foundation (C1), the foundation slab is assumed fully embedded in a thin layer of GSI material that is R (1) x = √Ab 휋 R (2) 휃 = 4 √ 16BL3 3휋 Istr = mstrHeff (3) 2 If = mf (4) ( R휃 2 4 + hf 2 3 ) Table 1 Stifness and damping coefcients for the soil-foundation system of each GSI confguration GSI Confguration (Section number) kx cx k휃 c휃 C1. GSI material around foundation (2.1) kbftfwflxflex cb + cs + cp kr훼휃 fembfl휃 fle휃 cr + cemb C2. Lateral disconnection only (2.2) kb(NS)ft cb(NS) kr(NS)훼휃 cr(NS) C3. GSI material below foundation with no embedment (2.3) kbflx cb kr훼휃 fl휃 cr C4. GSI material below foundation with lateral disconnection (2.4) kbftflx cb kr훼휃 fl휃 cr 3888 Bulletin of Earthquake Engineering (2023) 21:3881–3904 1 3 underlain by much stifer materials with shear wave velocity more than twice that of the GSI material. The full horizontal stifness model can be expressed as: where kb is the stifness due to the shearing action between the foundation base and the underlying soil. It is linearly proportional to the value of the secant shear modulus of the foundation material, Gsec, which is dependent on the material shear strain. The horizontal base stifness can be signifcantly reduced if the GSI material is much softer than the original natural soil (NS). For the GSI confguration based on lateral disconnection only (C2), without soil replacement, the foundation base is in direct contact with original NS, hence, the base stifness model is stated as kb(NS) in Table 1. Noted that 휈 is the Poisson’s ratio of the foundation material. ft considers the increased stifness as the foundation is placed at the bottom of the trench rather than at the ground surface, which is a function of the embedment depth, D, whilst fw takes into account the stifening efect due to the contacts between the embedded foundation walls and the surrounding material. The latter is not required when lateral disconnection is used (C2, C4). Both factors are not needed when there is no embedment (C3). Noted that Aw is the total contact area between the sides of the foundation and the surrounding material, which is equal to d × perimeter, in which d is the depth of efective contact depending on the condition of the backfll (e.g. contact is lost between backfll material and foundation/basement walls), whilst h is the depth of the centre of the total contact area measured from the ground surface. flx is a stifening factor when the GSI layer is underlain by the stifer original NS (C1, C3, C4), which is required when the value of shear wave velocity of the underlying layer is more than twice that of the surface GSI layer. However, flex is not required for C2, C3 and C4 as it is an additional stifening factor taking into account the embedment efect. Noted that HS is the total thickness of the GSI layer, measured from the ground surface to the bottom of the layer. A thinner GSI layer increases these two factors, and hence, the horizontal stifness, which would most probably reduce isolation efectiveness. kx = kbftfwflxflex (5) k (6) b = 8GsecRx 2 − 휈 f (7) t = 1 + 0.15√D B f (8) w = 1 + 0.52( h B ⋅ Aw L2 )0.4 flx = 1 + (9) 1 2 Rx HS (10) flex = 1 + 5 4 D HS Bulletin of Earthquake Engineering (2023) 21:3881–3904 3889 1 3 3.2 Horizontal radiation damping models (cx) For the benchmark GSI confguration (C1), the full horizontal radiation damping model can be expressed as: where 휌 is the density of the foundation materials. ( VLa) sec is the Lysmer’s Analog wave velocity as expressed below, which is similar to the compression-extension wave velocity that is controlled by the Young’s modulus of the material. cb is the contribution of radiation damping due to horizontal shearing actions at the foundation base that generate shear waves to the soil. This damping coefcient is linearly proportional to the value of the shear wave velocity of the foundation materials, whilst the larger the horizontal motions of the foundation, the larger amount of radiation damping can be resulted. Hence, an increase in foundation movements as parts of the GSI mechanism can help dissipate energy. For the confguration with lateral disconnection only (C2), without soil replacement, the foundation base is in direct contact with original NS, hence, the base radiation damping model is stated as cb(NS) in Table 1. cs is the coefcient that takes into account the radiation damping through the horizontal shearing actions at the two sides of the circumscribed rectangle, whilst cp considers the radiation damping based on the contact at the other two sides of the rectangle that are pushing back and forth against the soil, sending compression-extension waves into the half-space. These two coefcients are also linearly proportional to the respective wave velocity value. They are not needed when there is lateral disconnection (C2, C4) or when there is no embedment (C3). 3.3 Rocking stifness models (k) When the embedded foundation slab is surrounded by GSI material (C1), which is underlain by much stifer materials with shear wave velocity more than twice that of the GSI material, the full rocking stifness model can be expressed as: where (11) cx = cb + cs + cp cb = 휌 (12) ( VS ) secAb cs = 4휌 (13) ( VS ) sec(Ld) cp = 4휌 (14) ( VLa) sec(Bd) (15) ( VLa) sec = 3.4( VS ) sec 휋(1 − 휈) k휃 = kr훼휃fembfl휃fle휃 (16) 3890 Bulletin of Earthquake Engineering (2023) 21:3881–3904 1 3 in which, a0 is a dimensionless frequency factor defned by kr is the stifness due to the rocking restraint at the foundation base, which is linearly proportional to the secant shear modulus of the foundation materials. Parts of the GSI mechanism can be realised by the use of low-modulus foundation material that reduces the rocking stifness, such that rocking isolation can be achieved. For the confguration with lateral disconnection only (C2), the foundation slab is sitting on original NS, hence, the rocking stifness model is stated as kr(NS) in Table 1. 훼휃 is a frequency-dependent dynamic stifness modifer for rocking about y-axis at the base, which can be calculated based on the fundamental natural frequency of the SSFS system, 휔SSFS. femb is a factor accounting for the embedment efect on rocking motions, which is not relevant when there is lateral disconnection (C2, C4) or if there is no embedment (C3). fl휃 considers the stifening efect when the low-modulus GSI layer is underlain by the stifer original NS (C1, C3, C4). fle휃 is an additional stifening factor for the embedment efect when there is an underlying stifer layer, but this factor is not required if lateral disconnection is deployed efectively. A thinner GSI layer increases both factors, and in turn, the rocking stifness. 3.4 Rocking radiation damping models (c) For a foundation slab that is wholly embedded in low-modulus GSI materials (C1), the full rocking radiation damping model can be expressed as: where k (17) r = 8GsecR휃 3 3(1 − 휈) 훼휃 = 1 − (18) ⎡ ⎢ ⎢ ⎢ ⎣ 0.55a0 2 � 0.6 + 1.4 (L∕B) 3 � + a0 2 ⎤ ⎥ ⎥ ⎥ ⎦ (19) a0 = 휔SSFSB ( VS ) sec femb = 1 + 0.92 (20) (d L )0.6[ 1.5 + (d L )1.9(D d )0.6] fl휃 = 1 + (21) 1 6 R휃 HS (22) fle휃 = 1 + 7 10 D HS (23) c휃 = cr + cemb Bulletin of Earthquake Engineering (2023) 21:3881–3904 3891 1 3 in which, B휃 is the inertia ratio that can be calculated by in which, a frequency-dependent modifer can be calculated by cr is the radiation damping coefcient for foundation rocking, in which, Ib is the second moment of area of the foundation base about its centroidal y-axis. As the product 휌VLa is the seismic impedance of the foundation materials for the compression-extension wave, the expression is consistent in format with other radiation damping coefcients in the lumpedparameter model. This damping coefcient is linearly proportional to the value of the Lysmer’s Analog wave velocity of the foundation materials. While the low-modulus material for GSI would reduce this coefcient, the larger rocking motions of the foundation would increase the rocking radiation damping. For the GSI confguration based only on lateral disconnection without replacement of the original foundation soil (C2), the rocking radiation damping model is stated as cr(NS) in Table 1. cemb is the rocking radiation damping coefcient accounting for the three parts of the embedment efects. The frst term is about the shorter sidewalls (perpendicular to x-axis) rotating about y-axis of the foundation base pushing back and forth against the soil and emitting compression-extension waves, whilst the second term is about the same sidewalls shearing up and down and emitting shear waves. Finally, the third term is about the longer sidewalls (parallel to x-axis) rotating about y-axis of the foundation base generating torsional shear waves. All three damping components do not exist for GSI confgurations with lateral disconnection or when there is no embedment. 3.5 Equivalent properties of partially flled soil Polyurethane injection has been proposed as a promising GSI method for retroftting existing structures as discussed in Sect. 2.3 and 2.4. For cohesionless soils, injected polyurethane mixes with the soil, whereas for cohesive soils, polyurethane injection results in low-modulus polyurethane columns. In such cases, the total cross-sectional area of polyurethane can be estimated as a percentage of the foundation base area, PA, based on the centre-to-centre distance between injection holes, diameter of injection holes and the expansion coefcient of the polyurethane. Equivalent material properties of the modifed foundation soil layer can then be obtained for calculating the stifness and damping coeffcients. The same approach can be applied to other flling materials. cr = 휌 (24) ( VLa) secIb ( 1 1 + B휃 ) B (25) 휃 = 3(1 − 휈) 8 ( Istr + If ) 휌R휃 5 cemb = 4휌 (26) [( VLa) sec 3 d3B + ( VS ) secL2dB + ( VS ) sec 3 Ld( L2 + d2) ] ⋅ 휂 휂 = 0.25 + 0.65 (27) √a0 � d D �− a0 2 �D L �−0.25 3892 Bulletin of Earthquake Engineering (2023) 21:3881–3904 1 3 The secant shear modulus Gsec is the governing parameter for both horizontal and rocking stifness models as discussed in Sect. 3.1 and 3.3. The equivalent value, Geq, can be calculated from the shear modulus values of the original NS (GNS) and GSI material (GGSI): Likewise, the equivalent density, 휌eq, can be calculated from the density values of the original/modifed NS (휌NS) and GSI material (휌GSI): The equivalent shear wave velocity, ( VS ) eq, can then be obtained for calculating the horizontal and rocking radiation damping coefcients as described in Sect. 3.2 and 3.4. Noted that Gatto et al. (2022) have put forward a pair of equations for estimating the equivalent small-strain shear modulus and damping for the soil–polyurethane composite based on experimental results. 3.6 Response history analysis To design the various confgurations of GSI systems discussed in Sect. 2, response history analysis can be performed by solving the equations of motion based on the respective analytical design models proposed in this section, as a higher-tier approach compared to the hand-calculation method that is outlined in Sect. 4. The performance of the four GSI confgurations (C1–C4) is illustrated based on a fve-storey building, of which the details are provided in Sect. 4.1. The case study building is assumed sitting on a 0.5-m thick raft footing on the ground surface for C3 and embedded into the surface NS or GSI material for C1. As the lateral disconnection technique (C2, C4) is more efective for structure with deep basement (Somma et al. 2022), the case study building is modifed by adding a twolevel basement. The ground motion series recorded at the Tarzana station during the M6.7 Northridge earthquake in 1994 is used as an illustration (refer Sect. 4.2 for more details). The peak ground acceleration (PGA) is scaled to 0.5 g. The simulated response time series for the four GSI confgurations are plotted in Fig. 3, along with the respective reference cases. A lot of parameters can be obtained from the numerical simulations, but only the key parameters that control the dynamic soil-foundation-structure interaction behaviour are tabulated in Table 2 for further discussion. The horizontal and rocking stifnesses of the GSI foundation as compared to the respective reference cases govern the isolation efectiveness. The combined efect can be refected in the fundamental natural period of the fexible-base system, TSSFS (refer Sect. 4.6 for the equations). It is clearly seen that the percentage reduction of the structural demand is strongly correlated with the period lengthening ratio. It is understood that the frequency contents of the ground motions could have a signifcant impact on this. However, this is generally valid from the design perspective when multiple series of input ground motions are considered, especially for cases with the fexible-base natural period falling within the constant-velocity or constant-displacement ranges. Geq = GNS (28) ( 1 − PA ) + GGSIPA 휌eq = 휌NS (29) ( 1 − PA ) + 휌GSIPA (30) ( VS ) eq = √Geq 휌eq Bulletin of Earthquake Engineering (2023) 21:3881–3904 3893 1 3 Fig. 3 Response time series simulated based on the proposed analytical models for the four GSI confgurations (C1–C4), along with the respective reference cases Table 2 Horizontal and rocking stifnesses (kx and k휃), fundamental natural periods, TSSFS, as well as the peak absolute values of the total acceleration, ẍT str, of the case study structure with various GSI confgurations as compared to the respective reference cases in/on natural soil (NS) GSI confguration kx (MN/m) k휃 (MNm) TSSFS (s) ẍT str (g) C1 Embedded footing with GSI 212 7070 0.82 0.27 Ref. Embedded footing in NS 913 56,000 0.43 0.63 % diference −77% −87% +89% −56% C2 Two-level basement with GSI 819 44,200 0.61 0.37 Ref. Two-level basement in NS 2550 168,000 0.41 0.64 % diference −68% −74% +49% −42% C3 Surface footing with GSI 156 5250 0.99 0.24 Ref. Surface footing on NS 692 40,800 0.48 0.57 % diference −77% −87% +108% −58% C4 Two-level basement with GSI 81.2 4040 1.75 0.11 Ref. Two-level basement in NS 2550 168,000 0.41 0.64 % diference −97% −98% +324% −83% 3894 Bulletin of Earthquake Engineering (2023) 21:3881–3904 1 3 It is reasonable to consider that the relative values of the stifnesses between the GSI system and the respective reference case is critical for the system period shift ratio, as shown in Table 2. A closer inspection reveals that the absolute values of the stifnesses are also important. Taking the horizontal stifness as an example for discussion, lateral disconnection without the use of soft materials (C2) can reduce the stifness by 68%, which is comparable to the percentage reduction for C1 and C3; however, the system period is increased by 49% only, which in turn leads to a smaller demand reduction of 42%. This can be explained by using the classical system period equation of Veletsos and Meek (1974): As kstr equals 187 MN/m for the case study building, high foundation stifness values as for C2 (e.g. kx = 819 MN/m) would not enhance the system fexibility enough to signifcantly increase the system period. This explains the relatively smaller increase in the system period, even though lateral disconnection has signifcantly reduced the horizontal stifness by 68% and the rocking stifness by 74%. This fnding can also be illustrated by comparing between C1 and C3. While the reduction in stifnesses of the two confgurations are the same, the period shift ratio and hence the structural demand reduction are more signifcant for C3. That is because the stifnesses of surface footing are lower than those of embedded footing. It is noteworthy that the results presented herein are not indicative of the relative efectiveness of the four GSI confgurations, but to highlight the importance of the relative stifness between the structure and the foundation. More discussion about this can be found in Sect. 4.5. 4 Design procedure 4.1 Structural details The basic information of the structure is needed for the design of GSI system. The key parameters that control the seismic response of a fxed-base structure is the fundamental natural period, TFB, and the structural damping ratio, 휁str. The fundamental natural period can be estimated based on the efective height of the structure (typically measured from the ground surface), the structural form and the material of construction, as stipulated in various design codes or guidelines. The structural damping ratio can be estimated based on the material of construction, the design code level (e.g. ductility capacity) and the expected level of damage under the shaking level of the design earthquake scenarios. In addition, the efects of dynamic soil–structure interaction are afected by the dimensions of the foundation plan, the thickness of the raft footing or the depth of the basement, as well as the mass of the superstructure and the foundation/basement. As soft materials are placed underneath the GSI foundation system, the amount of ground settlement and wind-induced defection of the structure should be assessed against the required serviceability limits stipulated in design standards and guidelines. An ongoing study has indicated that a RSM layer of 2-m thick can support ordinary buildings of up to six to eight storeys without compromising serviceability requirements. Pile foundation can be used for taller or heavier buildings as investigated in Tsang et al. (2012a), whilst lower isolation efectiveness would be expected. T (31) SSFS = TFB √√√√1 + kstr kx + kstr( Heff )2 k휃 Bulletin of Earthquake Engineering (2023) 21:3881–3904 3895 1 3 This design case study considers a fve-storey reinforced concrete shear wall building with a total height of 15 m above ground level and a rectangular foor plan of 18 m by 12 m, which is consistent with the one adopted in recent investigations (Tsang and Pitilakis 2019; Tsang et al. 2021). The fundamental natural period of the fxed-base structure can be estimated as 0.37  s based on ASCE/SEI 7-22 and a 5% equivalent viscous damping is assumed. The lumped mass of the superstructure, mstr, is assumed as 650  Mg, whilst the lumped mass of the substructure, mf , is assumed to be 260  Mg for the 0.5-m thick raft footing or 580 Mg for the two-level basement. The lateral stifness of the fxed-base structure, kstr, can be calculated accordingly. The depth of the footing and the basement below ground, D, is 1 m and 9 m respectively. The efective height, Hef, of the structurefoundation system, measured from the base of the foundation, can be assumed as 11 m and 19 m, respectively, by considering a uniform distribution of mass along the height of the building. 4.2 Site hazard The earthquake action is characterised by the response spectrum of the design earthquake scenario or the design response spectrum that is stipulated in design code or guideline. It depends on the level of seismicity at the location of the structure, the distance from active faults, and the local subsoil and bedrock properties. In this design case study, the building is assumed located in a region of very high seismicity with a design PGA of 0.5 g and the analysis is performed based on the response spectrum recorded during the 1994 Northridge earthquake. The scaled response spectrum and the corresponding idealised design spectrum in the tripartite format are shown in Fig. 4a. 4.3 GSI confguration and material selection Firstly, the confguration of the GSI system has to be determined. The decision partly depends on the availability of space surrounding the structure to be isolated. If there is a limitation Fig. 4 a Response spectrum recorded during the 1994 Northridge earthquake scaled to a PGA of 0.5 g, and b the damping-corrected response spectra for the case study non-isolated (NS) and GSI systems 3896 Bulletin of Earthquake Engineering (2023) 21:3881–3904 1 3 to construct beyond the peripheral of the building foundation, the GSI layer can only be placed below the foundation (C3). Lateral disconnection (C2, C4) may still be possible as the required gap is small. If an existing structure needs to be retroftted, lateral disconnection without or with GSI material infll can be adopted, whilst injection techniques can also be explored to provide a base isolation layer. On the other hand, the choice of the low-modulus materials needs to be made and the dimensions of the GSI layer have to be designed. RSM as a GSI material has been well researched, hence, this is adopted in this design case study. Other low-modulus material, such as EPS geofoam, polyurethane, SAP, sand–SAP mixtures, can also be used. As GSI mechanism is primarily based on the lower stifnesses of the foundation materials, the small-strain shear modulus would be the governing design parameter. Sample values of various possible GSI materials collected from the literature (Alzawi and El Naggar 2011; Nappa et al. 2016; Tsang and Pitilakis 2019; Gatto et al. 2020) are provided in Table 3, given a confning pressure of 50–60 kPa, which is considered appropriate for the current case study, whilst the confning pressure for EPS geofoam and natural rubber is not available. Meanwhile, a set of the shear strain-dependent shear modulus reduction and damping curves for the material is needed. A 2-m thick layer of gravel–rubber mixtures (with 30% of rubber by weight) surrounding the raft footing (i.e. HS = 3 m) or the basement (i.e. HS = 11 m), both at the peripheral and below, i.e. confguration C1, is used for the GSI system in this design case study. For the same rubber content, gravel–rubber mixtures give lower small-strain shear modulus than sand–rubber mixtures. Other percentages of rubber in the mixture can be used, depending on the desirable stifness of the GSI layer. The use of pure rubber layers has also been found highly efective in reducing seismic demand in previous studies (Kaneko et al. 2013). A total thickness of 20  m is assumed for the entire stratum overlying bedrock (VR = 1800 m/s). The shear strain-dependent shear modulus reduction and damping curves for NS and RSM, as well as their small-strain values and the density, as presented in Tsang and Pitilakis (2019), based on the original studies of Anastasiadis et al. (2012) and Senetakis et al. (2012), are adopted in this design case study. The corresponding information for soil–polyurethane mixtures can be found in Gatto et al. (2022). 4.4 Surface shear strain estimation The GSI confgurations investigated in this paper rely on the reduced foundation stifnesses which are functions of the shear modulus. The nonlinearity of GSI materials is captured via the equivalent-linear approach, and hence, the level of shear strain in the region interfacing the foundation system needs to be estimated, such that representative values of the shear modulus and damping ratio can be obtained. The efective shear strain at the ground surface that controls the shear modulus and damping ratio of the surface stratum can be estimated by Eq. (32) as derived by Tsang and Pitilakis (2019) based on Tsang et al. (2006). in which, the factor R훾 is the ratio of the efective shear strain to the maximum shear strain induced during an earthquake, which can be assumed equal to 0.65; and u̇ g,max is the peak velocity of the input ground motions. � (32) s,eff = R� u̇ g,max ( VS ) sec 2� 1 + � √ � 1 − R4�4 Bulletin of Earthquake Engineering (2023) 21:3881–3904 3897 1 3 is the seismic impedance ratio, in which, VS and 휌S are the weighted-average shear wave velocity and density over the entire stratum overlying bedrock, including the surface GSI layer and the stifer natural soil sediments immediately underneath, while VR is the shear wave velocity of the underlying bedrock, which can be estimated empirically or by the resonant period equivalence (RPE) method (Tsang et al. 2012b), and 휌R is the corresponding density that can be estimated from the value of VR using the linear correlation given in Tsang and Pitilakis (2019). R is the refection coefcient defned as: 훽 is the energy dissipation factor based on the equivalent viscous damping, 휁eq, defned as: u̇ g,max can be calculated from the input ground motions, and is equal to 318 mm/s for the selected Northridge earthquake ground motions. If an acceleration design response spectrum is used, multiplying the PGA (in the unit of g) by 750 can provide a rough estimate of u̇ g,max (in the unit of mm/s), as adopted in AS 1170.4 (2007), based on Californian ground motion data. To estimate surface shear strain, iteration is required as ( VS ) sec, 훼, R and 훽 are shear strain-dependent. After a few iterations, the values of the key parameters in Eq. (32) calculated for the case study scenarios with embedded footing are summarised in Table 4. It is noted that, for the high design PGA level of 0.5 g, the surface shear strain is large, and the corresponding shear wave velocity would be signifcantly reduced by 50–70%. 4.5 Equivalent‑linear stifness and damping models Once the secant shear modulus of the GSI layer is calculated, based on the estimated level of efective shear strain as discussed in the previous sub-section, the equivalent-linear stifness and damping models, for both horizontal and rocking motions, for the corresponding GSI confguration can be obtained. Table 5 presents the values of each coefcient and factor for the design scenarios of GSI systems based on C1, in comparison with the respective reference scenarios. Cases with an embedded footing of 1 m deep and a two-level basement structure of 9 m deep, respectively, are considered. The diference of over 10 times between the secant shear modulus is directly refected in the stifnesses of the foundation base, kb and kr, given the linear proportionality between the shear modulus and the foundation stifnesses. This can facilitate dynamic soil–structure interaction, and therefore, enhance the isolation efect. It is, however, noted that the stifening efect (33) 훼 = 휌RVR 휌SVS R = (34) 1 − 훼 1 + 훼 훽 = exp (35) ( −휋휁eq) Table 3 Sample small-strain shear modulus Gmax (MPa) of various GSI materials Gravel-rubber (RSM) (30% rubber) Sand-SAP (33% SAP) EPS geofoam Polyurethane Natural rubber SAP Pure gravel (NS) (ref.) 5.9 32 6.6 25 0.6 0.41 120 3898 Bulletin of Earthquake Engineering (2023) 21:3881–3904 1 3 due to the much stifer material underlying the soft GSI layer, mainly represented by flx and fl휃, would inevitably narrow the diferences between the stifnesses. The damping coefcients are linearly proportional to the shear strain-dependent shear or Lysmer’s Analog wave velocity, which is, in turn, proportional to the square root of the secant shear modulus. This explains the smaller diferences in the damping coefcients between GSI systems and their respective references. On the other hand, the two-level basement structure has signifcantly elevated the stifnesses through factors fw and flex for horizontal stifness and femb and fle휃 for rocking stifness. As discussed in Sect. 3.6, this can have signifcant implications for the isolation efectiveness as the relative stifness between the structure and the foundation would control the period shift ratio, and in turn, the percentage of demand reduction. It is also seen that the damping coefcients, cs, cp and cemb, which are relevant to the embedment, are much higher as much more radiation damping is expected between the foundation walls and the surrounding materials. 4.6 Equivalent dynamic properties of fexible‑base system An exact solution for the damping of a fexible-base system, 휁SSFS, can be derived based on the approach proposed by Avilés and Pérez-Rocha (1996) and Maravas et al. (2014). Equation (36) explicitly includes the foundation inertia and the strain-dependent equivalent damping of foundation materials, which is a key parameter of an equivalent-linear fexible-base system. The exact solution is recommended because it retains the higher-order terms of all the damping components, which are signifcant for GSI systems. An upper limit of 30% is recommended herein for the total fexible-base system damping based on NIST GCR 12-917-21 (2012), which explains that empirical observations from case studies (Stewart et al. 1999) have shown an upper limit of approximately 25% for foundation damping (that excludes structural damping). in which, 휁x and 휁휃 are the horizontal and rocking radiation damping ratio, respectively, which can be calculated by Eqs. (37) and (38). 휁 (36) SSFS = mstr휁str kstr[1+4휁str 2 ] + (mstr+mf)(휁x+휁eq) kx [ 1+4(휁x+휁eq) 2 ] + (Istr+If)(휁휃+휁eq) k휃 [ 1+4(휁휃+휁eq) 2 ] mstr kstr[1+4휁str 2 ] + mstr+mf kx [ 1+4(휁x+휁eq) 2 ] + Istr+If k휃 [ 1+4(휁휃+휁eq) 2 ] (37) 휁x = cx 2 √ kx ( mstr + mf ) Table 4 Estimation of efective shear strain at the ground surface of the design case study scenarios with embedded footing ( VS ) sec (m/s) 훼 R 훽 훾s,eff  (%) GSI (C1) 32 38 −0.95 0.46 0.87 NS (ref.) 93 28 −0.93 0.53 0.32 Bulletin of Earthquake Engineering (2023) 21:3881–3904 3899 1 3 The associated exact solution for the fundamental natural period of a fexible-base system, TSSFS, is: It is noted that if the higher-order damping terms are negligible, Eq. (39) becomes: Moreover, if foundation inertia is insignifcant, Eq. (40) can be further reduced to the classical solution of Veletsos and Meek (1974), i.e. Eq. (31). (38) 휁휃 = c휃 2 √ k휃 ( Istr + If ) (39) TSSFS = 2휋 √ 1 + 4휁SSFS2 √√√√√ mstr kstr[ 1 + 4휁str2] + mstr + mf kx [ 1 + 4 ( 휁x + 휁eq)2 ] + Istr + If k휃 [ 1 + 4 ( 휁휃 + 휁eq)2 ] T (40) SSFS = 2휋 √ mstr kstr + mstr + mf kx + Istr + If k휃 Table 5 Equivalent-linear stifness and damping coefcients and factors for the case study GSI systems (C1), without and with basement structures, and the respective reference scenarios Design Scenario kx (MN/m) cx (MNs/m) k휃 (MNm) c휃 (MNms) Embedded footing with GSI kb 50.6 cb 8.3 kr 4340 cr 328 ft 1.06 cs 1.4 훼휃 0.63 cemb 181 fw 1.17 cp 1.6 femb 1.37 flx 2.38 fl휃 1.52 flex 1.42 fle휃 1.23 Embedded footing in NS (ref.) kb 573 cb 32.1 kr 49,100 cr 1335 ft 1.06 cs 5.4 훼휃 0.74 cemb 587 fw 1.17 cp 6.4 femb 1.37 flx 1.21 fl휃 1.08 flex 1.06 fle휃 1.04 Two-level basement with GSI kb 49.8 cb 8.2 kr 4274 cr 237 ft 1.18 cs 12.3 훼휃 0.63 cemb 1770 fw 1.99 cp 14.8 femb 3.30 flx 1.38 fl휃 1.14 flex 2.02 fle휃 1.57 Two-level basement in NS (ref.) kb 573 cb 32.1 kr 49,100 cr 1035 ft 1.18 cs 48.2 훼휃 0.73 cemb 6055 fw 1.99 cp 57.9 femb 3.30 flx 1.21 fl휃 1.08 flex 1.56 fle휃 1.31 3900 Bulletin of Earthquake Engineering (2023) 21:3881–3904 1 3 4.7 Structural demand calculation The damping and the fundamental natural period of the case study GSI systems (C1), without or with basement structures, and the respective reference scenarios can be calculated by using Eqs. (36) and (39) respectively. Those exact solutions are more suitable to be used because the value of each damping component is high, and so, the second-order damping terms would not be negligible. Also, the foundation mass would be signifcant for low-tomedium-rise buildings, to which GSI is expected to be applied, and especially so when basement structure exists. The results for the case study are summarised in Table 6. System period shift ratio is the determining factor for the isolation performance. It is seen that the GSI design adopted in this case study would increase the system period by around 70%. Even though the stifness models for the shallow footing of 1  m deep and those for the two-level basement of 9 m deep difer signifcantly, the system periods are still comparable. This applies to both non-isolated (NS) and GSI systems. This can be explained by the fact that when the embedment enhances the stifnesses, the foundation mass also increases. It is noteworthy that the classical system period equation, i.e. Eq. (31), that assumes insignifcant foundation mass, would not be able to capture this. Once the system damping is calculated, a damping correction factor can be applied uniformly across the entire period range of the response spectrum, for simplicity, to take into account the higher damping level of the soil-foundation-structure system. The value can be determined by Eq. (41) as stipulated in Eurocode 8 or Eq. (42) in ASCE/SEI 7-22. They give similar values for the whole range of damping ratio up to the recommended limit of 30%. A more accurate method is to compute the response spectrum at the exact level of damping as opposed to the default 5%. The damping-corrected acceleration response spectra (Sa) of both non-isolated (NS) and GSI systems are shown in Fig. 4b. This illustrates the application of the proposed design procedure. Only the cases with embedded footing are shown, as the values for the cases with a two-level basement are similar. The corresponding maximum base shear force (Vb) and the maximum drift (Δ) are also presented in Table 6. It can be seen that, for this design case study, the period shift efect alone can reduce the structural demand by 40–50%, whilst the enhanced damping would reduce the demand further by around 10–20%. 5 Closing remarks Buildings and infrastructure are expected to be resilient and sustainable, and this should be universal for humankind. To enhance afordability and accessibility, the usual approach is to modify existing advanced technologies into lower-class versions based on more economical choices of materials or manufacturing processes that usually result in compromised performance. GSI was put forward as a universal technology in the frst place. It attempts to re-evaluate the needs and capabilities of the community, to adapt to existing structural 휂 = (41) √ 0.1 0.05 + 휁SSFS 휂 = (42) 5.6 − ln( 100휁SSFS) 4 Bulletin of Earthquake Engineering (2023) 21:3881–3904 3901 1 3 confgurations and common construction routines, and to make the best use of precious raw materials through upcycling. The feld of GSI has promoted new opportunities for earthquake resistant design (Tsang 2022). Various GSI mechanisms, confgurations and materials have been proposed by researchers from around the globe in the past decade. Advanced numerical modelling, experimental testing and large-scale feld measurement have been conducted to understand the isolation mechanisms, investigate the use of various GSI materials and quantify their performance. To facilitate uptake and consistent use of this emerging technology, this paper intends to develop a suite of analytical design models for four GSI confgurations, which are all based on reducing foundation stifnesses. One of the four confgurations (C4) that combines lateral disconnection (C2) and the use of low-modulus materials below foundation (C3) is proposed for the frst time (Sect. 2). The analytical design models comprise of a formula for estimating surface shear strain and a set of equivalent-linear foundation stifness and damping models (Sect.  3), which vary amongst the four GSI confgurations (Table 1). A design procedure has been put forward (Sect. 4) that allows day-to-day engineering design without dynamic analysis or computer coding. As the foundation inertia and the foundation damping are signifcant for GSI systems, exact analytical solutions for the equivalent dynamic properties of fexible-base systems have been provided as part of the procedure. Applications of the analytical models for the four GSI confgurations have been illustrated through response history analyses and the hand-calculation design procedure has been demonstrated in detail. Funding Open Access funding enabled and organized by CAUL and its Member Institutions. The author has received research support from Swinburne University of Technology. Data availability The datasets generated during and/or analysed during the current study are available from the author on reasonable request. Declarations Competing interests The author has no relevant fnancial or non-fnancial interests to disclose. Open Access This article is licensed under a Creative Commons Attribution 4.0 International License, which permits use, sharing, adaptation, distribution and reproduction in any medium or format, as long as you give appropriate credit to the original author(s) and the source, provide a link to the Creative Commons licence, and indicate if changes were made. The images or other third party material in this article are included in the article’s Creative Commons licence, unless indicated otherwise in a credit line to the material. If material is not included in the article’s Creative Commons licence and your intended use is not permitted by statutory regulation or exceeds the permitted use, you will need to obtain permission directly from the copyright holder. To view a copy of this licence, visit http://creativecommons.org/licenses/by/4.0/. Table 6 Equivalent dynamic properties of the case study GSI systems (C1), without or with basement structures, and the respective reference scenarios, as well as the calculated structural demand Design Scenario 휁SSFS (%) TSSFS (s) Sa (g) Vb (MN) Δ (mm) Embedded footing with GSI (C1) 30 0.74 0.27 1.7 9.1 Embedded footing in NS (ref.) 18 0.43 0.62 3.9 21 Two-level basement with GSI (C1) 30 0.69 0.29 1.8 9.8 Two-level basement in NS (ref.) 15 0.41 0.71 4.5 24 3902 Bulletin of Earthquake Engineering (2023) 21:3881–3904 1 3 References Abdullah A, Hazarika H (2016) Improvement of shallow foundation using non-liquefable recycle materials. Jpn Geotech Soc Spec Publ 2(54):1863–1867 Alzawi A, El Naggar MH (2011) Full scale experimental study on vibration scattering using open and inflled (GeoFoam) wave barriers. Soil Dyn Earthq Eng 31:306–317 Anastasiadis A, Senetakis K, Pitilakis K (2012) Small-strain shear modulus and damping ratio of sand/rubber and gravel/rubber mixtures. 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