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SEISMIC ASSESSMENT OF SOIL STRUCTURE INTERACTION ON SEVERAL ISOLATED BRIDGE CONFIGURATIONS ADOPTING A PBEE METHODOLOGY

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ABSTRACT

The most recent development of earthquake engineering is based on concept of design consisting in prescribed performance rather than the more traditional prescriptive approaches. The paper aims to assess the effects of isolation devices and soil structure interaction on a benchmark bridge adopting a Performance-Based Earthquake Engineering methodology. Several isolated configurations of abutments and pier connections are compared performing the most representative isolation devices. Isolation systems suitability depends on many factors, mainly connected with ground effects. In this regards, the second purpose of this paper is to assess the effects of soil-structure interaction (SSI) on the studied bridge configurations. Contributions of isolation technique and soil structure interaction are assessed evaluating the resistance effects applied to Peak Ground Acceleration (PGA) levels in terms of cost and time repair quantities.

INTRODUCTION

Since Northridge Earthquake, research studies (see CALTRANS, 1994) have proved the significant role that soil structure interaction (SSI) can play during seismic excitations especially for bridges and therefore the necessity of incorporating SSI in their design. Despite that, limited number of studies have been available in literature focusing on SSI on seismically isolated bridge structures, such as Vlassis and Spyrakos (2001), Tongaonkar and Jangid (2003) and Ucak and Tsopelas (2008).

This lack is not due to insignificance of the problem, but it can be primarily attributed to complexity of the physical phenomenon and lack of design procedures. In particular, seismic isolation technique is based on uncoupling a structure from the damaging soil components by a mechanism that provides flexibility and energy absorption capacity at the same time. Both these benefits can be modified by soil deformability and energy dissipation in the ground.

The principal aim of this paper is to assess SSI role together with beneficial effects of isolation technique on a benchmark bridge representative of ordinary construction California highway bridges (Figure 1). For more details see Elgamal et al. (2011), Elgamal et al. (2012), Forcellini et al. (2012) and Mackie et al. (2012). Several configurations of abutments and pier column connections with isolation devices at the top of the column and at the abutment are tested with the goal to reduce bridge repair costs and time.

In particular, the method applied in the study is based on two consequent steps. First of all, analyses are performed with fixed-base conditions (SSI neglected). Then analyses on soils with increasing deformability are performed in order to take into account SSI effects on isolation technique in

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reducing the forces transmitted to columns and to abutments and consequently their damage. Results are performed adopting a Performance-Based Earthquake Engineering (PBEE) methodology that provides a useful framework able to understand the relationships among ground motion, superstructure and isolation system. The aim is to evaluate the ability of various design approaches and isolator system properties to reliably achieve targeted performance goals.

PBEE METHODOLOGY

PBEE methodology is based on the concept of design for prescribed performance rather than the more traditional prescriptive approaches, developed by the Pacific Earthquake Engineering Research (PEER) Center (http://peer.berkeley.edu). This methodology has seen rapid developments mainly with applications to buildings (FEMA-350, 2000; FEMA-356, 2000; ATC 58, 2007; ATC 63, 2007; TBI Guidelines Working Group, 2010). Recently, Mackie and co-workers have pioneered the development of a bridge performance-based analysis framework (Mackie et al., 2008, Mackie et al., 2010a, Mackie et al, 2010b) adopting the numerical PEER platform OpenSees (http://opensees.berkeley.edu, Mazzoni et al., 2009). Simultaneously, Elgamal and co-workers (Elgamal et al., 2011, Elgamal et al., 2012, Lu et al., 2010 and Lu et al., 2011) had embarked on development of OpenSeesPL (http://cyclic.ucsd.edu/openseespl), a graphical user interface for three-dimensional ground-foundation systems applying OpenSees as the finite element analysis engine.

In particular, PBEE methodology (Mackie et al., 2008 and Mackie et. Al, 2010a) aims to assess structural performances in terms of the probability of exceeding threshold values of socio-economic decision variables (DVs) in the seismic hazard environment. The PEER PBEE framework is fundamentally based on the application of the total probability theorem to disaggregate the problem into several intermediate probabilistic models that involve intermediate variables, such as repair items or quantities (Qs), damage measures (DMs), engineering demand parameters (EDPs), and seismic hazard intensity measures (IMs).

PBEE methodology consists of different steps. The first is the definition of Performance Groups (PGs), based on the association of the various structural and non-structural components, using the most common repair methods. Each PG contains a collection of components that reflect global-level indicators of structural performance and that contribute significantly to repair-level decisions. The notion of a PG allows grouping several components for related repair work; therefore PGs are not necessarily the same as the individual load-resisting structural components. The study considers the longitudinal drift ratio (PG1) and the relative longitudinal displacement between the deck end and the abutment (PG3), representing the column and the abutments damage respectively as the main contribution to the total repair costs and time, as shown in previous works (Elgamal et al., 2011, Elgamal et al., 2012 and Forcellini et al., 2012). More details on the whole PBEE methodology can be found in Mackie et al. (2008) and Mackie et al. (2010b), while numerical implementations inside the interface is descripted in Mackie et al. (2010a) and Lu et al. (2011).

The present work is based on a recently developed BridgePBEE user interface (Mackie et al. (2010b) and Lu et al. (2010)) here modified by introducing isolation devices models in order to assess effects of several isolated configurations of abutments and column connections. The study focuses on longitudinal behavior that might be provided by isolation devices. Responses are assessed in terms of repair cost and time quantities such as Crew Working Days (CWD) and total Repair Cost Ratio (RCR) defined as the ratio between the cost of repair and the cost of the new construction. This study can be considered one of the relatively few attempts that applies PBEE methodology in the bridge and infrastructure arena.

CASE STUDY

The investigate bridge is intended to be representative of the prevalent ordinary construction types for California highway, subjected to a typical seismicity. In this regard, some standard measures were taken into consideration: the bridge is a 90 m long, 2-span structure, supported on one circular column (1.22 m diameter) 12 m long, 6.70 m above grade (Figure 1). The deck is 11.90 m wide and 1.80 m deep and the weight is 130.30 kN/m. Each abutment is 25 m long with 30000 kN as total weight. Secondly, in order to reproduce typical California seismicity, the paper considers 10 selected ground motions taken from the PEER NGA database (http://peer.berkeley.edu/nga/). TABLE 1 shows the intensity measures (in terms of PGA) for each input motion in longitudinal, transversal and vertical direction (http://peer.berkeley.edu/svbin). PGA-cumulative distribution function (CDF) is shown in Figure 2 for the three directions. For more details, see Mackie et al. (2010) and Mackie et al. (2012).

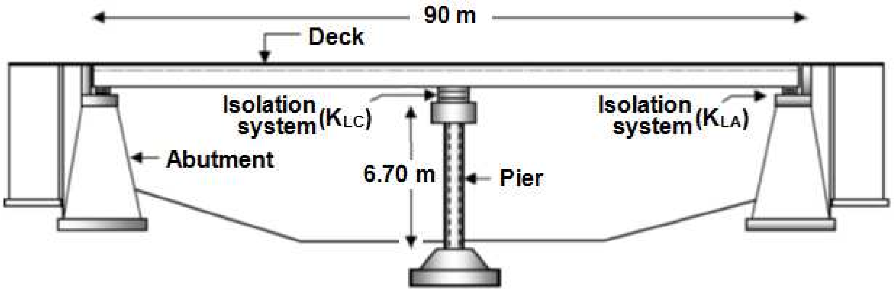

Figure 1 – Bridge case study

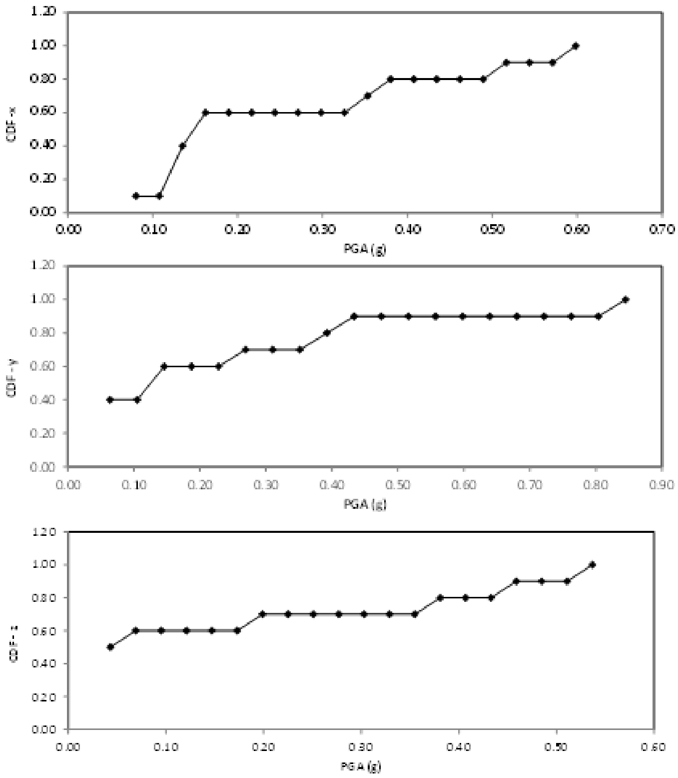

Figure 2 – CDF functions in three directions

Table 1 Input motions (PGA values)

Name	Date	Station	Long (g)	Trans (g)	Vert (g)
BORREGO A-ELC	4/9/1968 2:30	117 El Centro Array 09	0.130	0.057	0.030
LOMAP A2E	10/18/1989 0:05	58393 A PEEL 2E Haywars Muir Sch	0.171	0.139	0.095
LOMAP CAP	10/18/1989 0:05	47125 Capitola	0.529	0.443	0.541
NORTHR CNP	1/17/1994 12:31	90053 Canoga Park-Topanga Can	0.356	0.489	0.420
COALING H-PVP	5/2/1983 23:42	1162 Pleasant Valley P.P – bldg.	0.380	0.285	0.206
NORTHR SCS	1/17/1994 12:31	74 Sylmar – Converter Stat.	0.612	0.897	0.586
BORREGO B-ELC	10/21/1942 16:22	117 El Centro Array 09	0.068	0.044	0.033
COALINGA H-C05	5/2/1983 23:42	36227 Parkfiel – Colame 5W	0.147	0.131	0.034
IMPVALL H-CAL	10/15/1979 23:16	5061 Calipatria Fire Stat.	0.128	0.078	0.055
LIVERMOR A-KOD	1/24/1980 19:00	57187 San Ramon – Eastman Kodak	0.154	0.076	0.038

FEM MODEL

The Finite Element Model (FEM, Figure 3) was built with OPENSEES (Open System for Earthquake Engineering Simulation) that allows high level of advanced capabilities for modeling and analysing nonlinear responses of systems using a wide range of material models, elements, and solution algorithms (for more details see Mazzoni et al., 2009). In this regard, the reinforced concrete column is modeled with nonlinear forced-based elements (nonlinear beam - column) and fiber cross section, with 0.2 rad/m as the maximum curvature value at 11900 kN compression axial load. The deck is modeled with separate elastic beam-column elements (with cross Area of 5.72 m^2, transversal inertia 2.81 m^4 and vertical inertia 53.9 m^4). For more details, see Lu et al. (2011) and Mackie et al. (2012). The interface between the structure and the soil is modelled with specific element that allow to connect two separate points (one belonging to the structure and the second belonging to the soil), called equaldof. Soil is modelled with a 200 x 200 m, 25 m high 3D mesh made built with eight-node mixed volume pressure elements, called stdbrick. For all these details, see Mazzoni et al., 2009). The boundaries are modelled with periodic conditions using the penalty method (at any spatial location displacement degrees of freedom of the left and right boundary nodes were tied together both longitudinally and vertically). Thus, the base and lateral boundaries were modelled to be impervious, as to represent a small section of a presumably infinite (or at least very large) soil domain by allowing the energy imparted by the seismic event to be removed from the site itself. For more details, see Law et al. (2001), Elgamal et al. (2009) and Forcellini and Tarantino (2012). Soil model is developed within the framework of multi-yield-surface plasticity (Prevost, 1985) and focusing on controlling the magnitude of cycle-by-cycle permanent shear strain accumulation (Parra, 1996 and Yang et al., 2003) by specifying an appropriate non-associative flow rule (Prevost, 1985, Boushine et al. (2001), Dafalias (1986), Nemat-Nasser and Zhang (2002), Radi et al. (2002). In particular, the deviatoric component of the flow rule is associative, while non-associativity is restricted to the volumetric component only. Nonlinear shear stress-strain back-bone curve is represented by an hyperbolic relation, defined by low-strain shear modulus and ultimate shear strength constants. For more details see Yang et al. 2003, Elgamal et al. (2011), Elgamal et al. (2012).

In order to assess the effects of the supporting soil on the response of the entire system, behaviours of several isolated configurations are studied increasing soil deformability. First, a very hard soil with a sufficient high stiffness (if compared to isolator stiffness, as shown in Tongaonkar and Jangid (2003), simulating fixed conditions) is considered neglecting soil-structure interaction effects. Second, two cohesive soils with decreasing stiffness are taken into consideration. Table 2 shows the principal parameters for each soil model. The approach ramps make the connection with the longitudinal boundaries. Thanks to the high capabilities of the interface in implementing several support mechanisms at the abutments and at the top of the column, four bridge configurations have been compared, as described below.

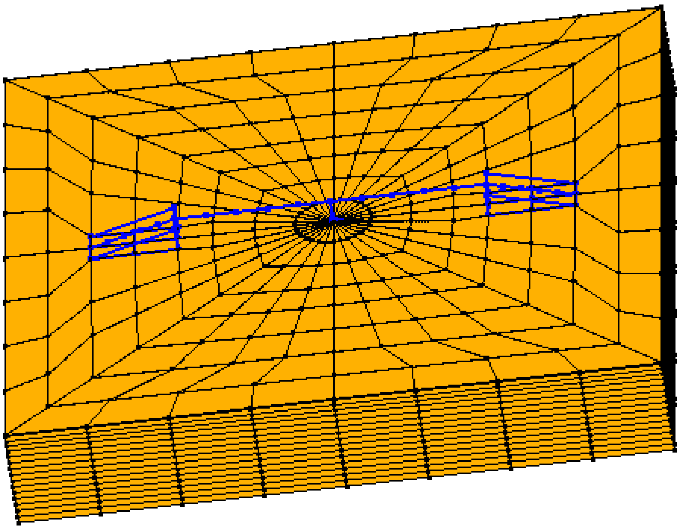

Figure 3 – Soil-Structure FEM model

Table 2 soil parameters

	FIXED	Medium Clay	Soft Clay
Mass density (t/ m^3)	2.0	1.5	1.3
Reference shear modul (kPa)	$3.7 \cdot 10^5$	$6.0 \cdot 10^4$	$1.3 \cdot 10^4$
Reference bulk modul (kPa)	$3.6 \cdot 10^6$	$3.0 \cdot 10^5$	$6.5 \cdot 10^4$
Cohesion (kPa)	180	37	18
Peak shear strain	0.1	0.1	0.1

ISOLATED CONFIGURATIONS

Isolation technique aims mainly at uncoupling the structure from the damaging effects of earthquake generally in the longitudinal direction. Its assessment has to take into account several aspects such as the suitability of a particular arrangement and the type of isolation system.

On the first hand, in this study the isolator devices are placed on the top of the column as well as at the abutments, and in order to evaluate the longitudinal resistance and behaviour only. The bearings are very stiff in vertical and transversal direction as specified below.

On the second hand, this study performs three types of isolation devices (elastomeric bearings, frictional/sliding bearings and roller bearings) that are developed and used in many countries all over the world (for more details see Kunde and Jangid, 2003). In particular, four isolated configurations were performed.

First of all, it was considered an original model (F-01) where abutments were isolated with two HDS 650x337s. They consist of two soft damping rubber bearing isolators, of soft compound with modulus of elasticity G=0.4 Mpa and equivalent viscous damping ξ=10% from ALGA S.p.A. (http://www.alga.it, Marioni, 2006). They are modelled with 2 simple elastic springs (730 kN/m each), as commonly used in professional bridge engineering applications.

This model is then compared with the case of a simple roller link connections between the deck and the abutments (F-02) that provides no resistance and thus it represents a stiff bridge (with a dominant period less than 1 sec, as shown in Forcellini et al., 2012).

The original fixed connection between the top of the column and the deck is then substituted with two different models of isolators that increase the flexibility of the bridge.

In the first model (HDRB-L), the isolation devices are two HDN 650x337s, consisting of two Normal Damping Rubber Bearing isolators with modulus of elasticity G=0.8 Mpa and equivalent viscous damping ξ=10% from ALGA S.p.A. (http://www.alga.it, Marioni, 2006). They are modelled with elastic springs (2920 kN/m; 2x1460 kN/m).

The second model (FP-NL) consists of two sliding pendulum devices belonging to ALGAPEND isolators on the top of the column and one on each abutment. They are made of HOTSLIDE series sliding materials, according to EN 15129 (European Standard on Antiseismic Devices) requirements and tested at the Eucentre laboratory at Pavia University. They are modelled with a simplified two-spring model, descripted in Kelly (1997), Kelly (2003) and Forcellini and Kelly (2013) that allows to consider both non-linear behaviour and buckling response using an explicit force–deformation relations. The two-spring model is composed by two rigid elements connected by moment springs across hinges at the top and bottom and by shear springs and frictionless rollers at mid-height. The kinematics of the model is described by two DOF (degree of freedom): the shear displacements and the relative rotation, for more details, see Kelly (1997), Kelly (2003) and Forcellini and Kelly (2013).

Figure 4 and Table 3 summarize the configurations adopted in this study.

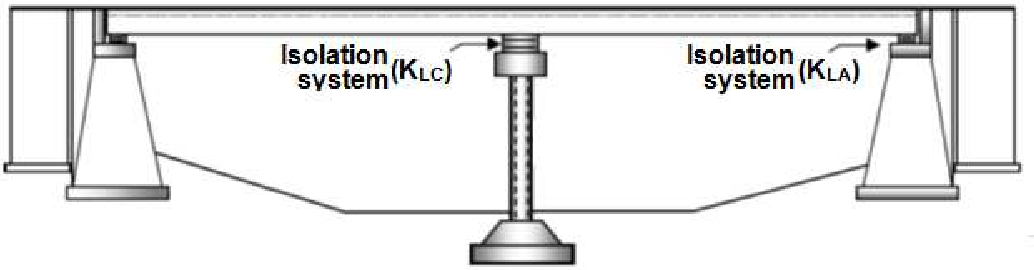

Figure 4 – Isolation details

Table 3 isolation details

	KLA (kN/m)	KLC (kN/m)	ISOLATION
F-01	730 x 2	Fixed	ABUTMENT ISOLATION
F-02	0	Fixed	NO ISOLATION
I-L	730 x 2	2920	FULL ISOLATION
I-NL	Spring Model	Spring Model	FULL ISOLATION

RESULTS

This paragraph compares fixed based responses with deformable soils (medium and soft clay) results for each configuration with increasing PGA values. Bridge main deformation consists of a longitudinal rigid translation that affects column and abutments damage.

Figure 5 shows isolation effects in reducing damage (and thus costs) in correspondence with column (PG1) for both linear (HDRB-L) and for non-linear (FP-NL) case. Linear isolation is instead seen to be ineffective for preventing damage at the abutments (PG3). On the contrary, non-linear isolation is able to take into account accumulation of hysteresis reducing damage (and thus costs) at the abutments. For more details, see previous works such as Forcellini et al. (2012) and Forcellini and Banfi (2013).

Figure 6 shows soil deformability effects for soft clay. Several considerations can be defined.

On the one hand, HDRB-L and F-01 responses can be compared with F-02 results whose behaviour is not affected by SSI, since the abutments are free to move. This comparison shows that linear isolation increases the damage both in the column (PG1) and in the abutments (PG3). Therefore, when linear isolation is considered, soil deformability gives non-conservatory designs and thus it should not be neglected, as pointed out by Ucak and Tsopelas (2008).

On the other hand FP-NL response is totally different from the other isolated linear configurations, thanks to non-linear behaviour of isolator devices that can preserve both the column and the abutment from damage reducing the final costs and time. In particular, if compared with F-02 model results, non-linearity is seen to reduce PG3 contribution and thus abutments damage. This reflects non-linearity effect in reducing the deformations (and consequently the damage) transferred to the abutments.

Figure 7 shows FP-NL results case in correspondence with the three soil models. Soil structure interaction affects expected costs connected to PG1 and PG3 in different ways. Column damage is affected for all PGA range with a reduction in the expected costs. Abutments damage shows a significant reduction between 0.55 g and 0.75 g depending on soil deformability. For example if PGA value is set around 0.65 g, the expected PG3 costs for fixed conditions are 20000 $, around the 33% if compared with soft case.

Non linear isolation works properly at low values of PGA, where it is possible to reduce abutments damage especially in correspondence with deformable soils. This behaviour is clearly shown considering total repair costs and time response. In particular, effects become non conservative for big values of PGA (more than 0.60g for RCR and more than 0.45g for RT). In these case, repair costs and time are bigger for soft and medium cases than those resulted from fixed case. Therefore, SSI effects cannot be neglected in design considerations. On the contrary, for smaller values of RCR and RT, neglecting SSI leads to conservative designs and in these cases, fixed hypothesis for soils can be considered conservative.

Moreover, other findings can be considered, for example, total costs and time are affected mainly by the abutment damage and thus saving the column does not necessarily means reducing the total costs and time for the bridge, as described in Forcellini and Banfi (2013). This is due to the fact that the damage is moved to the abutments especially with deformable soils.

Finally, while linear and non linear isolation have similar effectiveness in reducing column damage, they have completely different behaviours regarding abutments damage for both fixed based and deformable soils. Therefore, linear modelling of isolator commonly used in professional design cannot be considered sufficiently accurate to realistically assess bridge isolation behaviours. In this regards, the study assesses the benefit of non-linear isolators in protecting structural elements and thus reducing their damage, especially when based on deformable soils.

CONCLUSIONS

The study conducted in this paper may be viewed as an original contribution to seismic assessment of a simple single column bridge abutment configuration, taking into consideration performance aspects in terms of repair costs and time. In particular, isolation technique and soil structure interaction were studied applying a Performance Based Earthquake Engineering approach based on the disaggregation of results into performance groups.

Isolation technique is evaluated considering the most representative isolation devices (elastomeric bearings, sliding pendulum and roller bearings) modelled with the most credited theories. Secondly a parametric study on soil deformability was performed in order to assess the circumstances under which soil structure interaction need to be considered.

Results show the importance of two main considerations. First of all, soil structure interaction can affect negatively bridge final performance if not properly modelled. Secondly, the paper assesses non-linear isolators effects in protecting structural elements and reducing damage, especially at the abutments.

In particular, the paper shows how taking account SSI can sometimes become non conservative and thus the importance of considering its effects in design procedures. Contrasting what resulted from Vlassis and Spyrakos (2001) and Tongaonkar and Jangid (2003), SSI was shown to be significant even when simple linear behaviour for isolation devices is considered. Similar conclusions were pointed out by Ucak and Tsopelas (2008), even if without considering performance implications.

Therefore, adopting a performance approach was fundamental to consider economic considerations and thus this study can be considered one of the relatively few attempts to consider the seismic performance of isolated bridge configurations aiming at contributing to new easy-to-use design procedures for engineers all over the world.

Further analysis will aim to reproduce transversal response for the structure and highly non linear models for the isolators, both aspects that can significantly modify the seismic response.

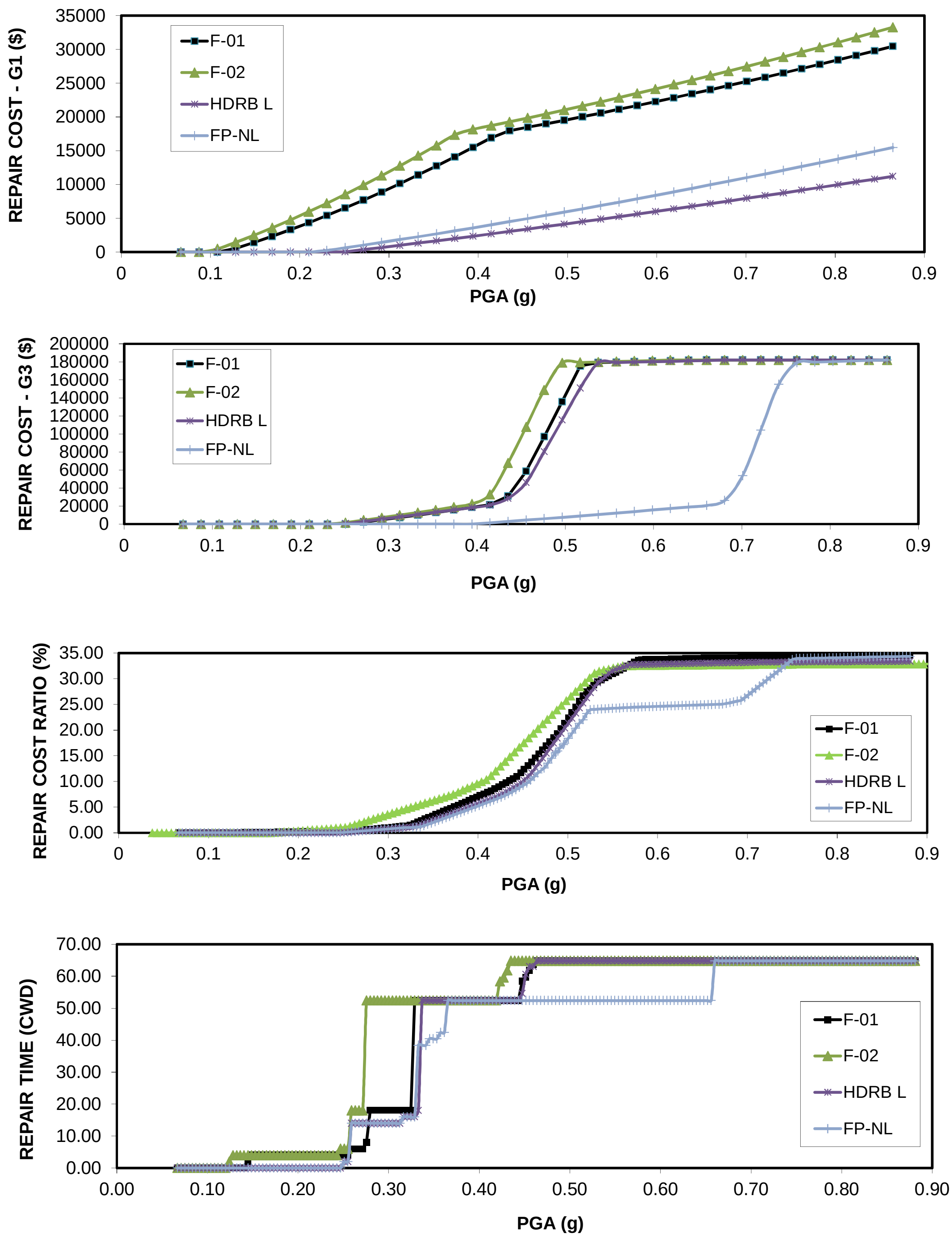

Figure 5 – Fixed case results

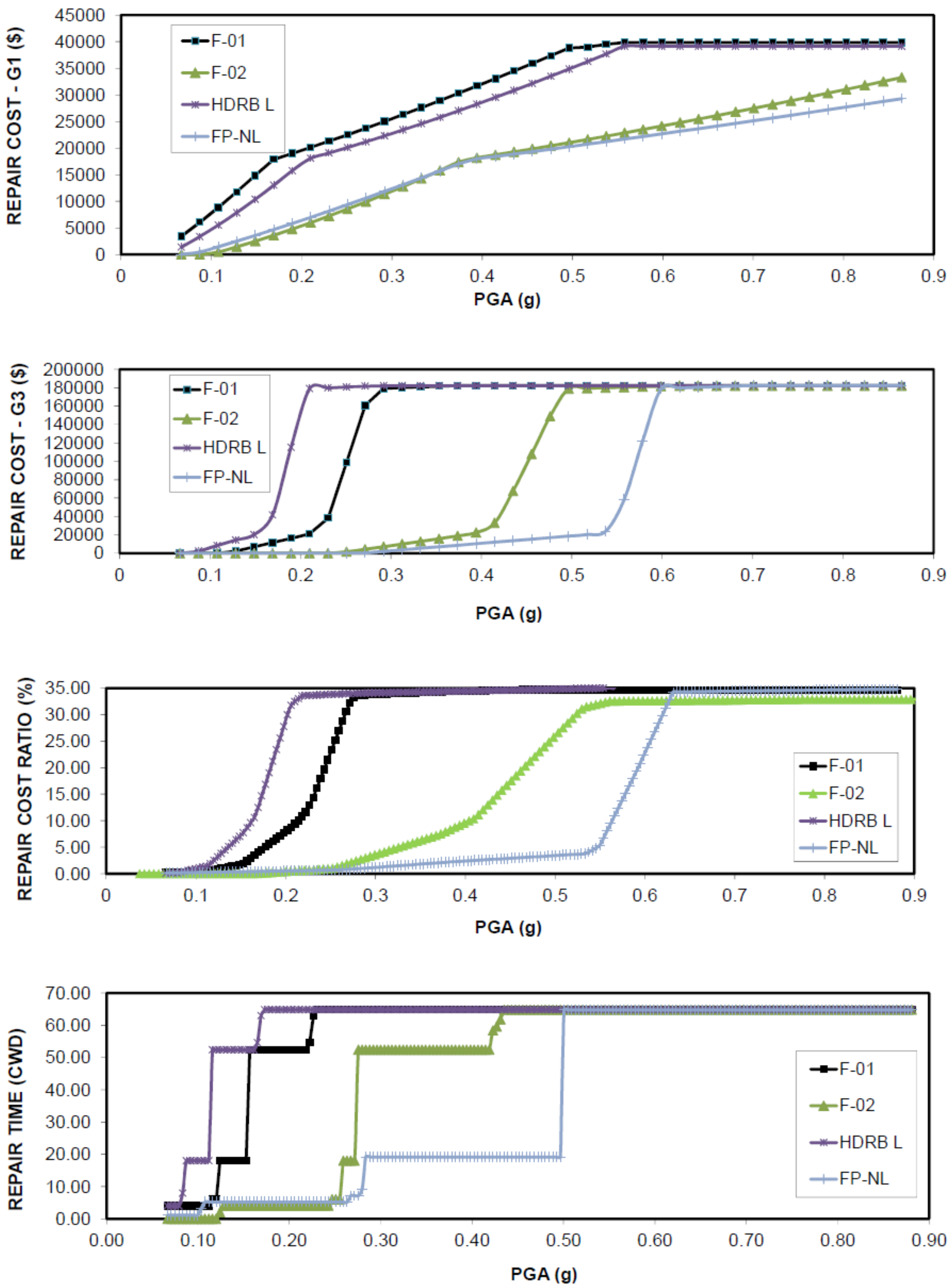

Figure 6 – Soft soil results

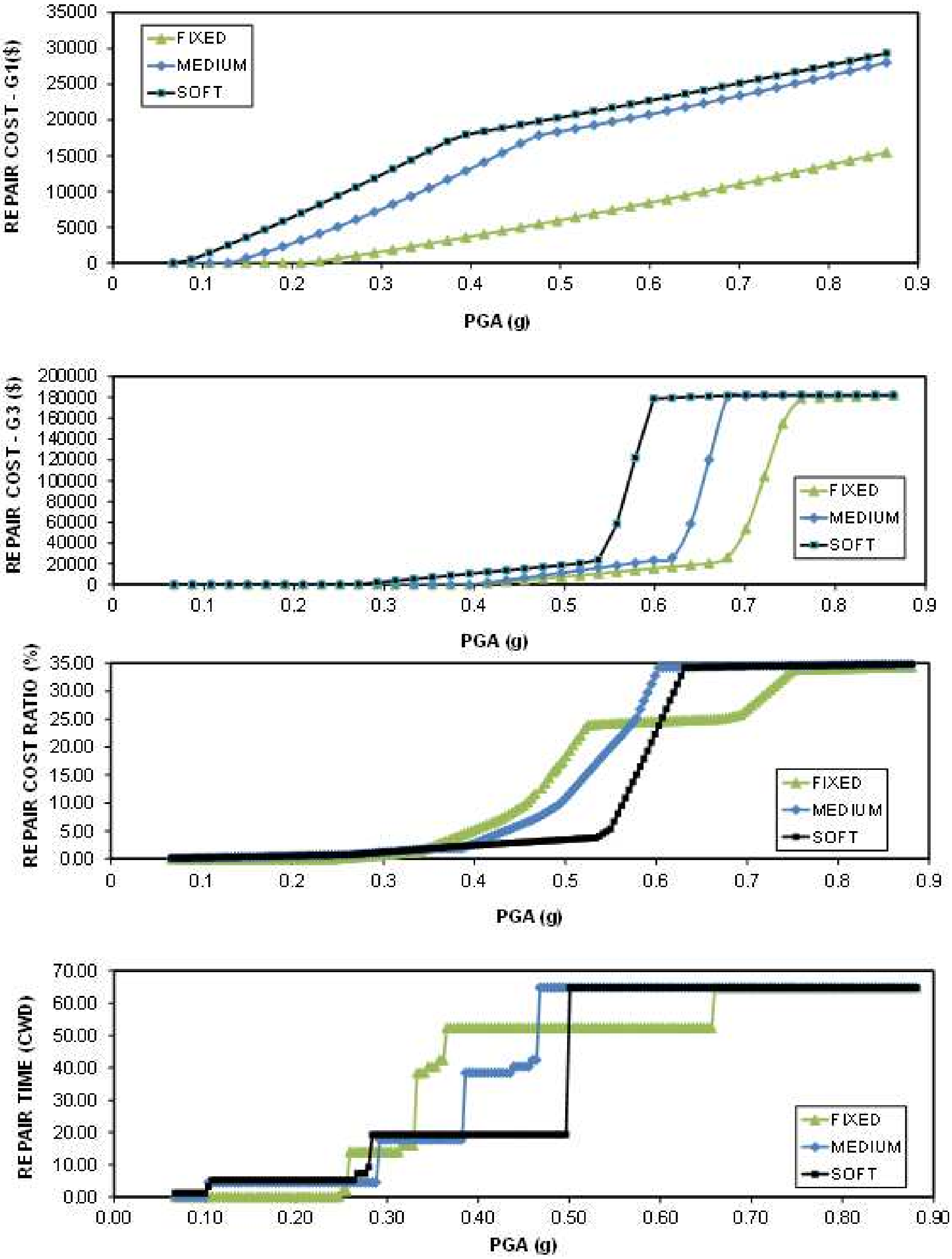

Figure 7 – FP-NL results

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