EVALUATION OF THE EFFECT OF PGV/PGARATIO OF STRONG GROUND MOTIONS ONRESPONSES OF SOIL STRUCTURE SDOFSYSTEMS

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ABSTRACT

Generally, in order to evaluate the seismic demand of structures, it is assumed that the structure is located on a rigid soil. However, with increasing the soil flexibility there will be significant variations in the structural response, i.e. the effects of Soil-Structure Interaction (SSI). Furthermore, in the near-field zone, pulse-like motions play crucial roles in the design of structures. This paper addresses the effects of Peak Ground Velocity to Peak Ground Acceleration ratio (PGV/PGA) of near-fault ground motions as a compound intensity index that can describe the frequency characteristics of ground motion on response of various soil-structure SDOF systems. A total 49 near-field ground motions records were selected which have been classified into two categories: first, records with a strong velocity pulse, (i.e. forward-directivity); second, records with a residual ground displacement (i.e. fling-step).

Parametric studies between PGV/PGA ratio of pulse-like ground motions and maximum relative displacement (U_{max}) indicate that with increase in structure-to-soil stiffness ratios($\bar{S}$), earthquakes with higher PGV/PGA ratio produce greater responses. Moreover, increasing in slender ratios ($\bar{h}$) and decreasing in mass ratios ($\bar{m}$) result in the responses of soil-structure SDOF systems become greater in all structure-to-soil stiffness ratios.

INTRODUCTION

In the near-field zone, pulse-like motions play crucial roles in the design of structures. In order to find a dependable intensity measure for design of civil structures, much effort has been devoted to analysis and to evaluate seismic performance of various systems subjected to such excitations. In some studies, the role of high ground velocities was accentuated to the extent that peak ground velocity (PGV) is often considered as the effective indicator of damage potential (Hall etal., 1995). On the other hand, some other studies indicated that acceleration pulses are in general leading engineering demand parameters for most civil structures than velocity pulses (Makris and Black, 2004).

It is worth to note that, Although PGA and PGV are very useful intensity measures for seismological studies, none of them can provide any information on the frequency content or duration of the motion. While, there is the agreement among the researchers concerning the influences of frequency content on seismic responses of civil structures. Consequently, PGA and PGV have to be completed by additional information for the proper characterization of a ground motion (Kramer, 1996). The ratio of PGV to PGA (ratio) is a ground motion parameter which provides information about frequency content of the input motion. Since PGA and PGV are usually associated with motions of different frequencies (Kermani et al., 2009). Moreover, in pulse-like ground motions, the coherent long-period pulses may lead to the PGV/PGA ratio of ground motions become larger (Alavi and Krawinkler, 2001). Hence, the PGV/PGA ratio is a very important parameter to characterize the damage potential of near-fault ground motions and indicated as being a measure of destructiveness (Consenza and Manfredi, 2000). The ground motions with higher PGV/PGA values have larger damage potential (Meskouris, 1992).

Moreover, it should be noted that the seismic analysis of engineering structures is often conducted based on an assumption that the structure is founded on a rigid semi-space, while in most situations the structures are supported by soil deposits.There is lack of studies in which the SSI phenomenon is considered in near-field ground motions with different features versus far-field records (Davoodi et al., 2012)

In this study, a parametric study is conducted to present a relationship between the maximum relative displacement of soil-structure SDOF systems and PGV/PGA ratios of wide range ofnear-field ground motions.

SOIL-STRUCTURE MODEL

The SSI effects rely on the properties of both structure and supporting soil, which may alter widely. Although different models can be adapted to consider SSI effects, a linear soil-structure SDOF system with a longer natural period and mostly a higher damping ratio can be employed as a simple model(Wolf, 1985).

In present study, the effects of SSI have been investigated on elastic response of linear surface structure which is subjected to horizontal seismic excitations. A simplified discrete model as shown in Fig. 1 is used to represent the real soil-structure system.

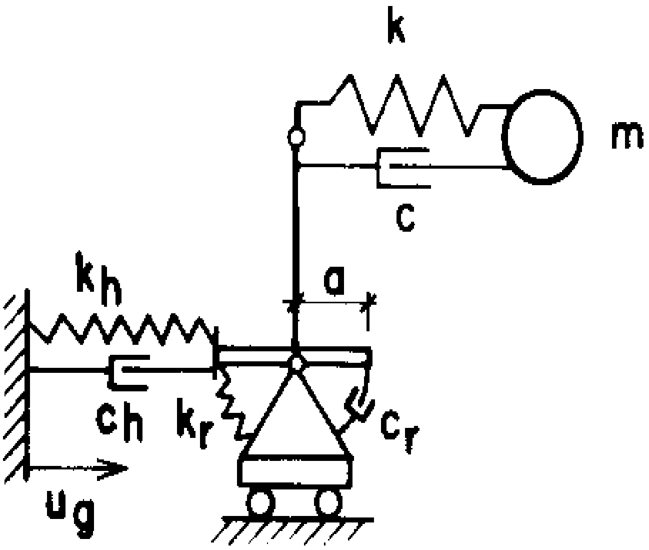

Figure 1. Soil-structure model (Wolf, 1985)

This model is based on the following assumptions:

- An equivalent linear SDOF system introduced as a structure.
- A massless circular rigid disk applied as a foundation.
- The soil beneath the foundation is considered as a homogeneous half-space.

EQUATIONS OF MOTION OF EQUEVALENT SDOF SYSTEM

For a specific excitation, the response of a dynamic system relies on the characteristics of the structure relative to those of soil. The following dimensionless parameters can describe the effect of SSI effectively Table1.

Table 1.SSI non dimensional parameters

Parameters	Definition	Assigned values
$\bar{S} = \frac{\omega_s h}{V_s}$	Structure-to-soil stiffness ratio	0.1, 1, 5
	(ω_s: fixed base frequency of structure, V_s: shear-wave velocity of the soil, *h*: height of the structure)	
$\bar{h} = \frac{h}{a}$	Slenderness ratio	0.5, 1, 5
$\bar{m} = \frac{m}{\rho a^3}$	Mass ratio of the structure to foundation	0.5, 3, 10
	(ρ:mass density of the soil)	
ϑ	Poisson's ratio of the soil	0.33
ζ_g, ζ	Hysteretic material damping ratios of the soil and the structure	0.05, 0.025

To consider SSI effect, a SDOF system must be replaced with an equivalent system which has higher hysteretic damping ratio "$\tilde{\zeta}$" and less natural frequency "$\tilde{\omega}$",Eq.(1, 2)(Wolf, 1985).

$$\tilde{\omega}^2 = \frac{{\omega_s}^2}{1 + \frac{\bar{m}\bar{S}^2}{8}\left(\frac{2-\vartheta}{\bar{h}^2} + 3(1-\vartheta)\right)} \tag{1}$$

$$\tilde{\zeta} = \frac{\tilde{\omega}^2}{{\omega_s}^2}\zeta + \left(1 - \frac{\tilde{\omega}^2}{{\omega_s}^2}\right)\zeta_g + \frac{\tilde{\omega}^3}{\omega_s^3}\frac{\bar{S}^3\bar{m}}{\bar{h}}\left(0.0036\frac{2-\vartheta}{\bar{h}^2} + 0.028(1-\vartheta)\right) \tag{2}$$

In this study, the equations of motion for equivalent one-degree-of-freedom system with a rigid basement were derived in the time domain, Eq. (3).

$$u + 2\tilde{\zeta}\tilde{\omega}\dot{u} + \tilde{\omega}^2 u = -\frac{\tilde{\omega}^2}{\omega_s^2}\, u_g \tag{3}$$

Where, "u" is lateral displacement of mass.

GROUND MOTION DATABASE

In this study, the ground motion database compiled for numerical analyses consists of a large number of near-field ground motions to cover a range of frequency content, duration, and amplitude. Near-field records are classified based on the presence of forward-directivity effect and fling-step effect. Moreover, near-field ground motions recorded within 30 km. In two sub-data sets the assembled database can be investigated. The first set contains 15 near-field ground motions characterized with forward-directivity effect and is divided into normal and parallel component records given in Tables 2 and 3. The second set includes 19 near-field ground motions records characterized with fling-step effect given in Table 4.All the time histories are recorded on soil classified as type C or D according to the NEHRP site classification. Selecting these soil conditions is required to consider SSI effects properly. The whole ground motion records were extracted from PEER Strong Motion Database of Berkeley University (PEER, 2006).

VERIFICATION OF THE ANALYSIS PROCIDURE

In this section, the code verification is conducted through the only available corresponding reference which was presented by Wolf in 1985.The digitized artificial time history normalized to 0.1g as same as Wolf procedure, and then applied to base of soil-structure SDOF system (Figure 1). The maximum of the relative displacement "U_{max}" is plotted as a function of the stiffness ratio ($\bar{S}$) for three different fixed-base structural frequencies. The results of these two studies are plotted in Figure 2 simultaneously.As can be seen, the trend of figures presents satisfactory correspondence with each other; however, there are quantitative discrepancies.

Table 2. The characteristics of near-field ground motions with forward-directivity effect (The normal component)

No.	Earthquake	Year	Station	M_w	Dist. (km)	PGA (g)	PGV (cm/s)	PGD (cm)
1	San fernando	1971	Pacoima Dam-Left Abutment	6.61	11.86	1.45	115.66	30.46
2	Gazli	1976	Karakyr	6.8	12.82	0.599	64.94	24.18
3	Coyote lake	1979	Gilroy Array #6	5.74	4.37	0.452	51.53	7.09
4	Coalinga	1983	Pleasant Valley P.P. - bldg	6.36	9.98	0.377	32.37	6.45
5	Morgan hill	1984	Anderson dam(Downstream)	6.19	16.67	0.449	29.01	3.91
6	Nahanni, Canada	1985	Site1	6.76	6.8	0.853	43.82	16.08
7	N. Palm Springs	1986	North Palm Springs	6.06	10.57	0.669	73.55	11.87
8	Whittier Narrows-01	1987	Santa Fe Springs - E.Joslin	5.99	11.73	0.398	23.75	1.76
9	Superstition Hills-02	1987	Parachute Test Site	6.54	15.99	0.418	106.74	50.54
10	Loma Prieta	1989	Gilroy Array #2	6.93	29.77	0.406	45.65	12.53
11	Sierra Madre	1991	Cogswell Dam-Right Abutment	5.61	18.77	0.297	15.01	2.05
12	Erzican, Turkey	1992	Erzican	6.69	8.97	0.486	95.4	32.09
13	Northridge-01	1994	LA dam	6.69	11.79	0.576	77.09	20.1
14	Kobe	1995	KJMA	6.9	18.27	0.854	95.75	24.56
15	Chi-Chi	1999	TCU065	7.62	26.67	0.831	129.55	93.85

Table 3. The characteristics of near-field ground motions with forward-directivity effect(The parallel component)

No.	Earthquake	Year	Station	M_w	Dist. (km)	PGA (g)	PGV (cm/s)	PGD (cm)
1	San fernando	1971	Pacoima Dam-Left Abutment	6.61	11.86	0.827	34.43	18.67
2	Gazli	1976	Karakyr	6.8	12.82	0.71	71.05	24.7
3	Coyote lake	1979	Gilroy Array #6	5.74	4.37	0.333	27.14	4.48
4	Coalinga	1983	Pleasant Valley P.P. - bldg	6.36	9.98	0.284	19.02	2.47
5	Morgan hill	1984	Anderson dam(Downstream)	6.19	16.67	0.276	29.52	6.44
6	Nahanni, Canada	1985	Site1	6.76	6.8	1.17	36.53	4.36
7	N. Palm Springs	1986	North Palm Springs	6.06	10.57	0.615	29.2	3.52
8	Whittier Narrows-01	1987	Santa Fe Springs - E.Joslin	5.99	11.73	0.51	33.09	4.16
9	Superstition Hills-02	1987	Parachute Test Site	6.54	15.99	0.343	49.57	21.78
10	Loma Prieta	1989	Gilroy Array #2	6.93	29.77	0.302	27.58	6.11
11	Sierra Madre	1991	Cogswell Dam-Right Abutment	5.61	18.77	0.261	9.19	0.85
12	Erzican, Turkey	1992	Erzican	6.69	8.97	0.419	45.29	16.52
13	Northridge-01	1994	LA dam	6.69	11.79	0.415	40.74	16.01
14	Kobe	1995	KJMA	6.9	18.27	0.548	53.38	10.27
15	Chi-Chi	1999	TCU065	7.62	26.67	0.557	82.27	55.05

Table 4. The characteristics of near-field ground motions with fling-step effect

No.	Earthquake	Year	Station	Comp.	M_w	Dist. (km)	PGA (g)	PGV (cm/s)	PGD (cm)
1	Chi-Chi	1999	TCU074	EW	7.6	13.75	0.59	68.9	193.2
2	Chi-Chi	1999	TCU074	NS	7.6	13.75	0.37	47.95	155.4
3	Chi-Chi	1999	TCU084	EW	7.6	11.4	0.98	140.43	204.6
4	Chi-Chi	1999	TCU129	EW	7.6	2.21	0.98	66.92	126.1
5	Kocaeli	1999	Yarimca	EW	7.4	3.3	0.23	88.83	184.8
6	Kocaeli	1999	Sakarya	EW	7.4	3.2	0.41	82.05	205.9
7	Chi-Chi	1999	TCU102	EW	7.6	1.19	0.29	84.52	153.9
8	Chi-Chi	1999	TCU089	EW	7.6	8.33	0.34	44.43	193.9
9	Chi-Chi	1999	TCU049	EW	7.6	3.27	0.27	54.79	121.8
10	Chi-Chi	1999	TCU067	EW	7.6	1.11	0.48	94.31	181.3
11	Chi-Chi	1999	TCU075	EW	7.6	3.38	0.32	111.79	164.4
12	Chi-Chi	1999	TCU076	EW	7.6	3.17	0.33	65.93	101.7
13	Chi-Chi	1999	TCU072	NS	7.6	7.87	0.36	66.73	245.3
14	Chi-Chi	1999	TCU072	EW	7.6	7.87	0.46	83.6	209.7
15	Chi-Chi	1999	TCU065	EW	7.6	2.49	0.76	128.32	228.4
16	Chi-Chi	1999	TCU079	EW	7.6	10.95	0.57	68.06	166.1
17	Chi-Chi	1999	TCU078	EW	7.6	8.27	0.43	41.88	121.2
18	Chi-Chi	1999	TCU082	EW	7.6	4.47	0.22	50.49	142.8
19	Chi-Chi	1999	TCU128	EW	7.6	9.08	0.14	59.42	91.05

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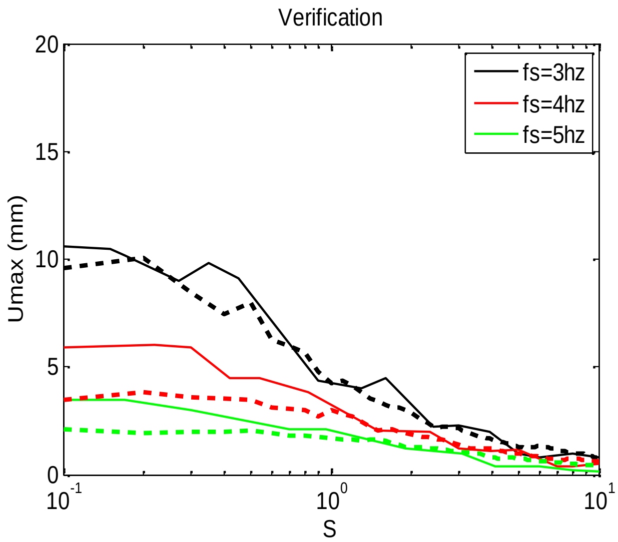

Figure 2. Maximum displacement responses of soil-structure SDOF system due to an artificial time history, ($\bar{h}$=1,$\bar{m}$=3,ϑ=0.33, ζ=0.025 and ζ_g=0.05), (a) relative displacement, (b) total displacement, (Noted the results of Wolf study and this study are presented by solid and dashed lines, respectively)

PARAMETRIC STUDY

The main goal of this section is to present the relationship between the PGV/PGA ratio, as a compound intensity index, and maximum relative displacement of varied soil-structure SDOF system caused by pulse-like ground motions in three different stiffness ratios ($\bar{S}$=0.1, 1, 10). The results are presented in Fig. 3, 4, 5 in which each straight line is the result of linear curve fitting.

Figure 3 shows that increasing in stiffness ratios result in the maximum distortion of structures subjected to ground motion with higher PGV/PGA ratios (typically pulse type ground motions with high period pulse) becomes predominant (Sadjadi, 2012).

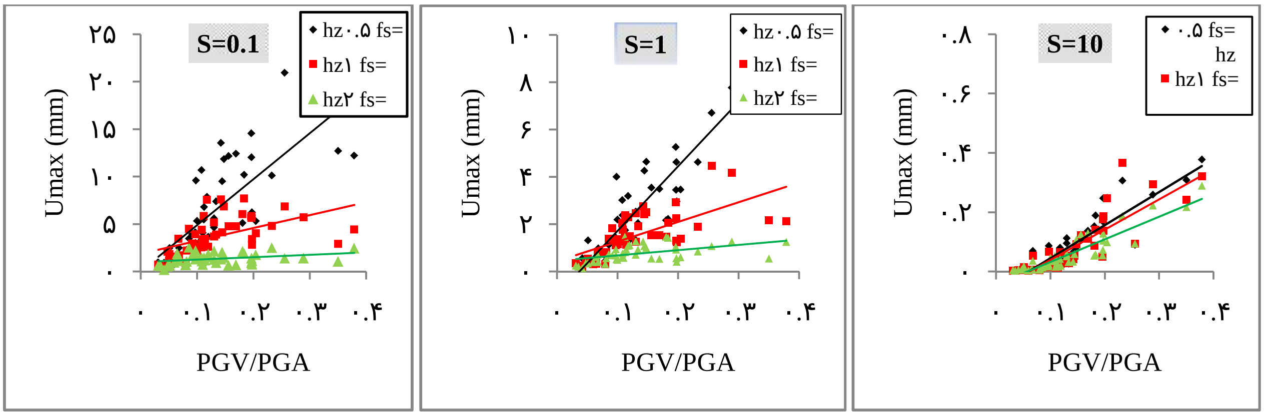

Figure 3.Relationship between the relative displacement of soil-structure SDOF system and PGV/PGA ratio ($\bar{h}$=1,$\bar{m}$=3,ϑ=0.33, ζ=0.025 and ζ_g=0.05)

As can be seen in Fig. 4, at low structure to soil stiffness ratio (S=0.1), structural responses with different slenderness ratiosare mostly close to each other. However, with increase in stiffness ratios ground motions with higher PGV/PGA ratio produce higher responses relative to ground motions with lower PGV/PGA ratios.

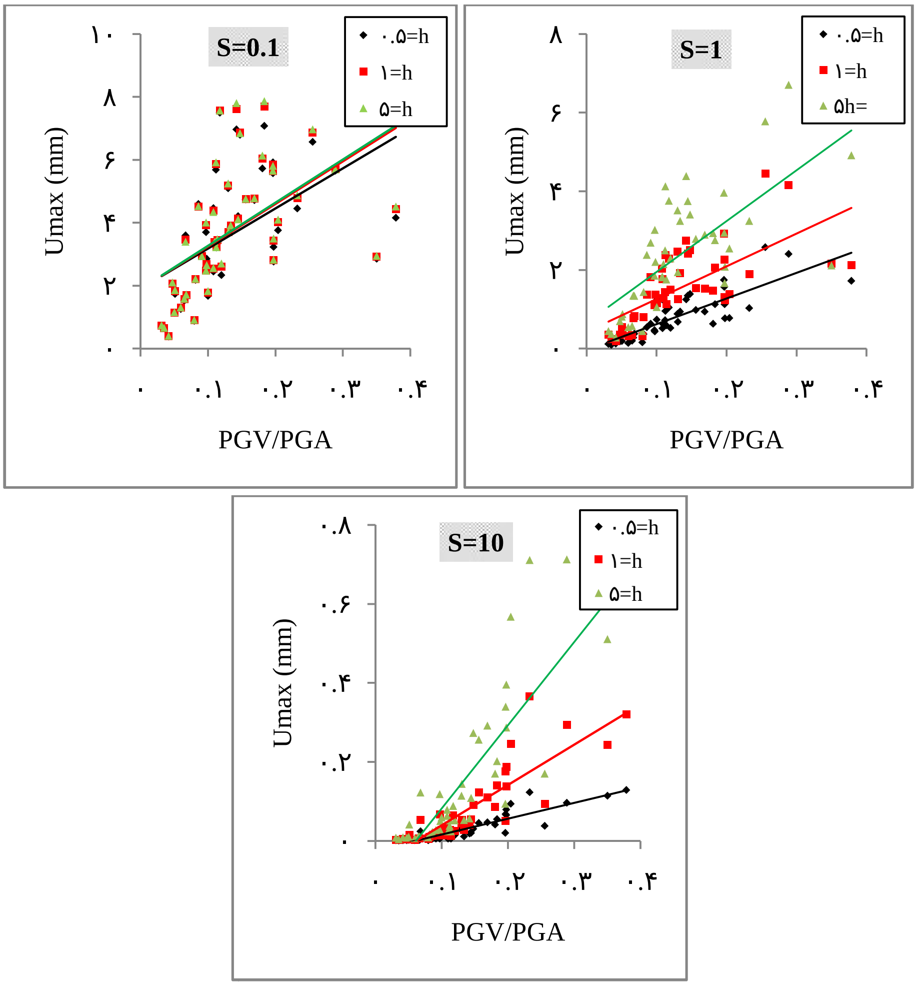

Figure 4.Relationship between the relative displacement of soil-structure SDOF system and PGV/PGA ratio ($\bar{m}$=3,fs=1,ϑ=0.33, ζ=0.025 and ζ_g=0.05)

As an illustration, Fig. 5 shows that at low stiffness ratios, mass ratios of the structure to foundationdo not have significant effects on responses of soil structure SDOF system. With decrease in soil stiffness relative to structure, structures with lower mass ratios, undergo higher distortion result from all pulse type near fields records which bear different PGV/PGA ratios.

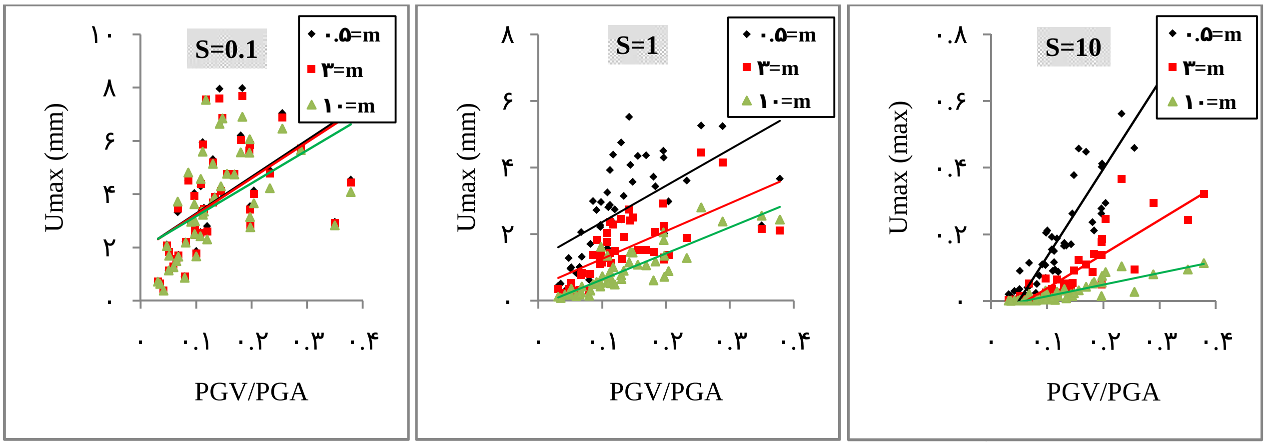

Figure 5.Relationship between the relative displacement of soil-structure SDOF system and PGV/PGA ratio ($\bar{h}$=1,f_s=1,ϑ=0.33, ζ=0.025 and ζ_g=0.05)

CONCLUSIONS

In this paper, the maximum elastic displacement responses of soil–structure systems produced by a large number of near-field ground motions were studied parametrically for assessing the effects of SSI. The results show that:

1. Earthquakes with higher PGV/PGA ratios tend to produce greater maximum dynamic responses at higher stiffness ratios than the ones with lower ratios.
2. At low structure to soil stiffness ratio (S=0.1), structural responses with different slenderness ratiosare mostly close.
3. With decrease in soil to structurestiffness ratios, structure with lower mass ratios undergo higher distortion result from all records with different PGV/PGA ratios.

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