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DYNAMIC BEHAVIOR OF A SMART BUILDING SUPPORTED WITH DAMPERS CONSIDERING SOIL-STRUCTURES INTERACTION

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Abstract- The paper introduces a mathematical model that determines the dynamic response of a structure retrofitted with viscous fluid dampers (FD), taking into account its P-Delta effect and soil-structure interaction (SI). The model reduces the dynamic-response of the internal forces of beam-columns when the structure is induced by an earthquake. The study includes numerical examples of two steel buildings, one with FD and SI, and the other without SI. The results highlight the difference between the two structures and provide valuable insights for civil-engineering structural designers to better resist seismic loading.

Keywords - Dynamics of Structures, Structural Control, P-Delta analysis, Viscous Fluid Dampers, Soil-structure Interaction

I. INTRODUCTION

The use of dampers to enhance seismic resistances is recently popular over the world thanks to the efficiency of seismic resistances. Viscous fluid dampers are one of the most useful passive devices, its reasonable and economical [4][5][22].

The superstructure of a building is deeply examined such as the shear frame model (SFM) which considers beams accompanied by slabs as rigid bodies and the finite element method (FEM) which considers beam and column flexural stiffness and their axial stiffness. However, SFM is not proper for structures with large spans. FEM does not consider local soil conditions, and regional geology beneath its structure, the effect of pile group or distance between two piles, or the effect of axial load in a beam-column element on its flexural stiffness [6][7][8][9][10][11]. Hence, both p-delta and SI analyses for a FD structure provide a more exact dynamic response than FEM analysis.

Dynamic properties of a structure be governed by on its natural periods which are affected by soil-structure interaction (SI), and by its beam-column flexural stiffness (P-Δ effect). The research of SI covers several approaches such as Winkler model [13], Direct Method [12][14], or the simplest method-Lump parameter model [15][16][18].

To more reasonably evaluate the efficiency of dynamic response reduction of FD structures, a computational model of FD building considering P-Delta effects for beam-column elements and SI could be analyzed.

II. THE MODEL OF FD STRUCTURES CON P-DELTA AND SI ANALYSIS

II.1. COMPUTATIONAL MODEL

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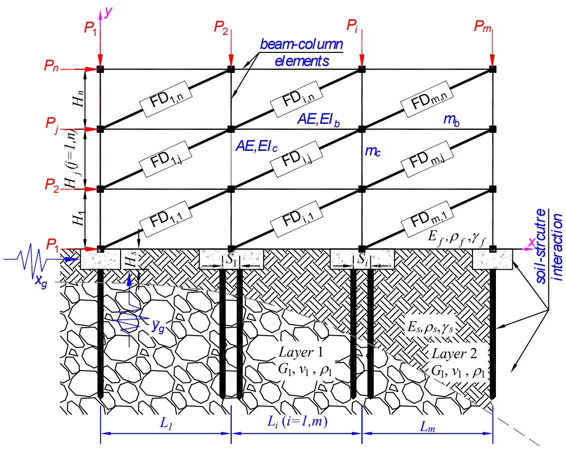


Fig. 1: A FD structure with SI

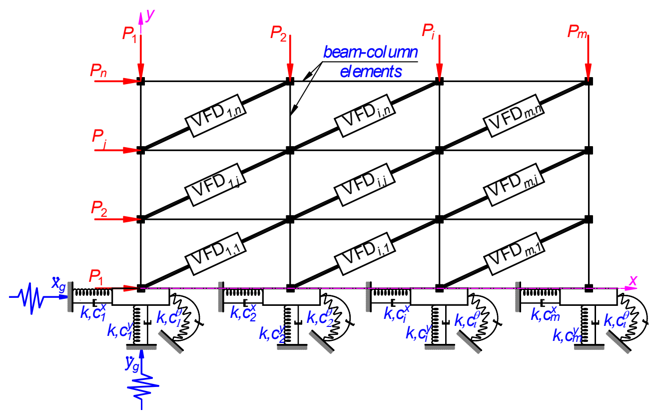


Fig. 2. Mathematical model of a structure retrofitted with FD subjected to external dynamic forces

Consider the m-bay, n-story planar frame and its pile foundation shown in Fig. The structure employs ($m\times n$) FD equipment at each of the portals. The excitation consists of n lateral forces P_j and horizontal and vertical earthquake loadings $\ddot{x}_g, \ddot{y}_g$. Flexural stiffnesses of the beams and columns are $EI_{i,j}^b$ and $EI_{i,j}^c$, respectively. The beam-column stiffness matrix is obtained as [11]

$$\mathbf{K_e} = \frac{EI}{L}\begin{bmatrix} \frac{A}{I} & 0 & 0 & -\frac{A}{I} & 0 & 0 \\ & \frac{2(s_1+s_2)}{L^2} & -\frac{(s_1+s_2)}{L} & 0 & -\frac{2(s_1+s_2)}{L^2} & -\frac{(s_1+s_2)}{L} \\ & & s_1 & 0 & \frac{s_1+s_2}{L} & s_2 \\ & & & \frac{A}{I} & 0 & 0 \\ & & & & \frac{2(s_1+s_2)}{L^2} & \frac{(s_1+s_2)}{L} \\ sym & & & & & s_1 \end{bmatrix} \quad (1),$$

where

$$s_1 = \sum_{n=0}^{6} a_n \rho^n \quad \begin{cases} a_6 = -0.00056103 \\ a_5 = 0.0022397 \\ a_4 = -0.0082555 \\ a_3 = 0.035493 \\ a_2 = -0.17009 \\ a_1 = 1.316 \\ a_0 = 4 \end{cases}$$

and

$$s_2 = \sum_{n=0}^{6} a_n \rho^n \quad \begin{cases} a_6 = 0.00054729 \\ a_5 = -0.0021273 \\ a_4 = 0.0073511 \\ a_3 = -0.027864 \\ a_2 = 0.10051 \\ a_1 = -0.329 \\ a_0 = 2 \end{cases}$$

with $\rho = \frac{PL^2}{\pi^2 EI} \in [-2, 2]$

From the above assuming and using Da Lambert principle, the differential equation governing the motion of a structure equipped with FDs is expressed in matrix form as $\mathbf{M\ddot{u} + C\dot{u} + Ku = P - M\,l\,\ddot{u}_g - F_{VFD}}$ (2), where $\mathbf{M}$ is the consistent or lump mass matrix. $\mathbf{K}$ is a global stiffness matrix including the stiffness of soil-pile foundation $\mathbf{K_{SI}}$ determined as [12] [15] and of beam- column elements $\mathbf{K_{CnB}}$ determined as [1]; and $\mathbf{C}$ is the damping matrix computed using the Rayleigh formula as [2]. $\mathbf{u}$ is a displacement vector; $\dot{\mathbf{u}} = \frac{d}{dt}\mathbf{u}$ and $\ddot{\mathbf{u}} = \frac{d^2}{dt^2}\mathbf{u}$ are velocity and acceleration vectors; $\mathbf{P} = [P_1, ..., P_i, ..., P_n]^T$ is an external force vector; $\mathbf{l}$ is a diagonal one matrix; $\ddot{\mathbf{u}}_\mathbf{g} = \begin{Bmatrix} \ddot{x}_g \\ \ddot{y}_g \end{Bmatrix}$ is ground acceleration; $\mathbf{F_{VFD}}$ is a damping force vector generated by FD [4]. value of $\mathbf{F_{VFD}}$ derives from the manufacture and does not exceed the maximum damper force [5].

II.2. NUMERICAL METHOD FOR COMPUTATION OF MOTION EQUATION

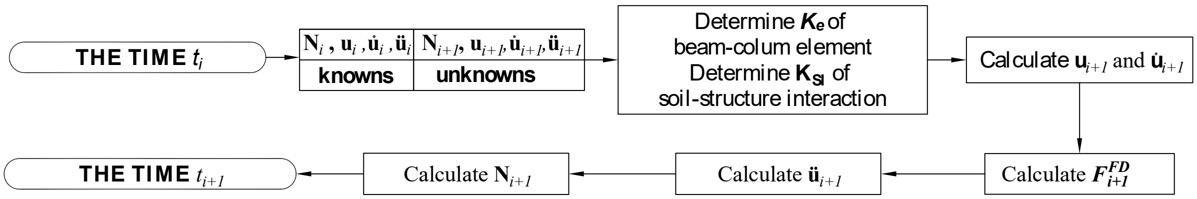


Fig. 3. Algorithm of one-time interval

Due mostly to non-linear forces generated from FDs and elastic forces from beam-column elements of geometry nonlinearity, equation (2) in the time domain is resolved using the modified Newmark method. The time domain is divided to obtain discrete constant values of t_i and t_{i+1} at every Δt. The response at the time instants t_{i+1} depend on not only applied loads but also the preceding quantities of axial forces at the time t_i. The numerical method for equation (2) is illustrated in Fig with the help of MATLAB routine.

III. NUMERICAL EXAMPLES

The 9-story steel building [23] retrofitted with FDs has yield strength σ_y=345MPa and the damping ratios for two first modes of ζ_1=ζ_2=2%. Its dynamic properties are given in Fig. 4. The first three natural periods of the structure are $T_1 = 1.20s$; $T_2 = 0.49s$; and $T_2 = 0.33s$. Building foundations are of two kinds I and II. Foundations I are at exterior corner columns and foundations II are at interior columns. The concrete grade for foundations is M350 (TCVN) [24] with E_p=30Gpa. The diameter of piles is $2R_p$=0.4m. The number of piles in foundation II is $n_p = 3\times 3 = 9$ with the ratio of S (distance between two piles) and $2R_p$ as $\frac{S}{2R_p} = 5$. The number of piles in foundation I is five with the distance between two piles of S as well.

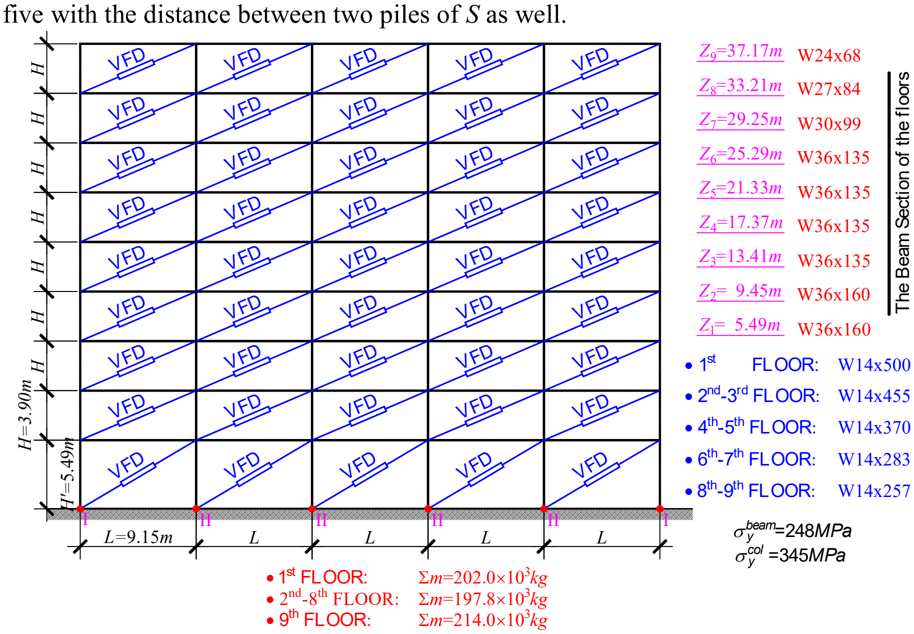


Fig. 4. The Benchmark 9-story building

Layer	TABLE-1: **Soil at pile cap**					
	Height H_s^{cap} (m)	Young modulus E_s^{cap} (MPa)	Poisson coeff. ν_s^{cap}	Shear modulus G_s^{cap} (MPa)	Density ρ_s^{cap} (kg/m^3)	Soil velocity c_s^{cap} (m/s)
I	2.5	30.0	0.3	11.5	1835	79
	Soil at piles					
I	35	30.0	0.40	10.7	1937	74
II		50.0	0.35	18.5	1937	98
III		70.0	0.25	28.0	1937	120

IV		90.0	0.20	37.5	1937	139

Therefore, the stiffness and damping of foundations I and II are [16][17][19][20][21][25] as $2k^{I}_{x,y} = k^{II}_{x,y} = 1685.1\times10^{3}\ kN/m$, $2c^{I}_{x,y} = c^{II}_{x,y} = 18756\ kN.s/m$, $2k^{I}_{\theta} = k^{II}_{\theta} = 28179\times10^{3}\ kN/m$, $2c^{I}_{\theta} = c^{II}_{\theta} = 81392\ kN.s/m$

The ElCentro earthquake [3] acts on the building along the x axis with peak ground acceleration (PGA) of $\left(\ddot{x}_g\right)_{\max} = 0.35g$ (Fig. 5). Analysis duration is 35 seconds with constant time intervals of Δt=0.00125s. The response of the structure are analyzed into two groups of non-controlled and FD-controlled structures as TABLE-2 with the FD in one portal as $C^{VFD}_{j} = 2\times10^{6}\ Ns/m; \alpha_j = 1; f^{VFD}_{j,\max} = 60kN$

TABLE-2: ANALYZED CASES

Name of cases	Analysis combined			
	Linear	P-Δ	SI	FD
(LIN without SI)$_{NCT}$	✓			
(LIN with SI)$_{NCT}$	✓		✓	
(P-Δ without SI)$_{NCT}$		✓		
(P-Δ with SI)$_{NCT}$		✓	✓	
(LIN without SI)$_{FD}$	✓			✓
(LIN with SI)$_{FD}$	✓		✓	✓
(P-Δ without SI)$_{FD}$		✓		✓
(P-Δ with SI)$_{FD}$		✓	✓	✓

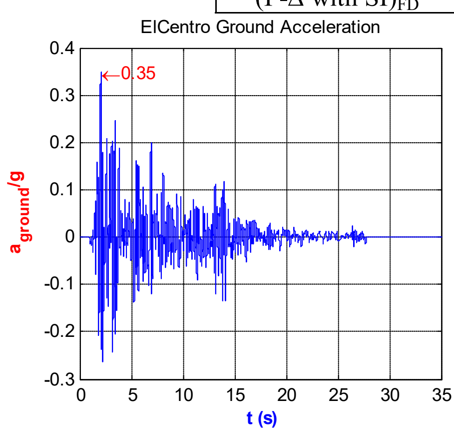


Fig. 5. Time history of the ElCentro ground acceleration [3]

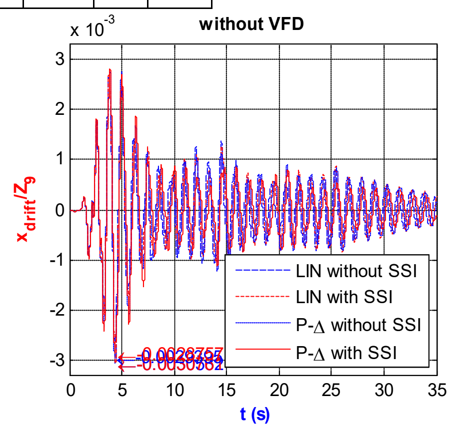


Fig. 6. Story drift response versus time without FD

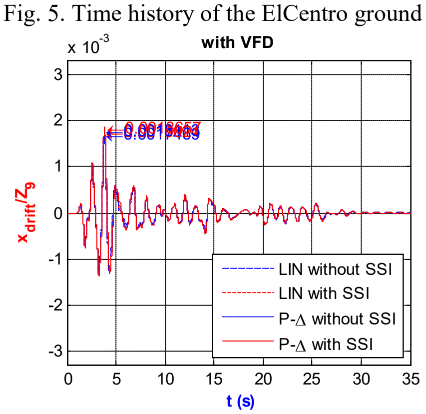


Fig. 7. Story drift response versus time with FD

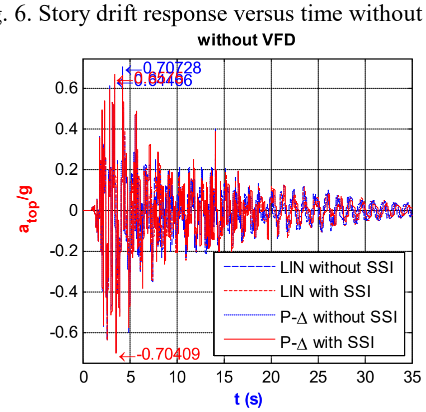


Fig. 8. Top acceleration response versus time without FD

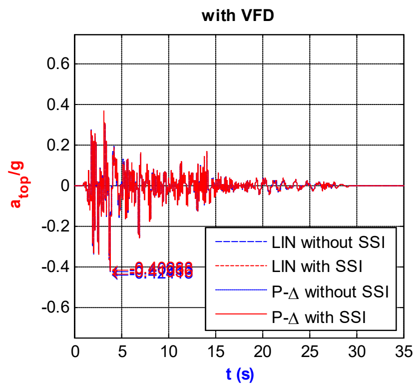


Fig. 9. Top acceleration response versus time with FD

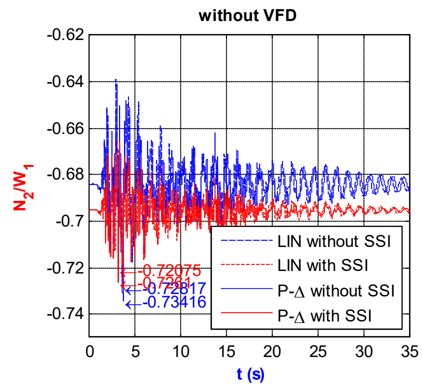


Fig. 10. Axial force of the second column without FD

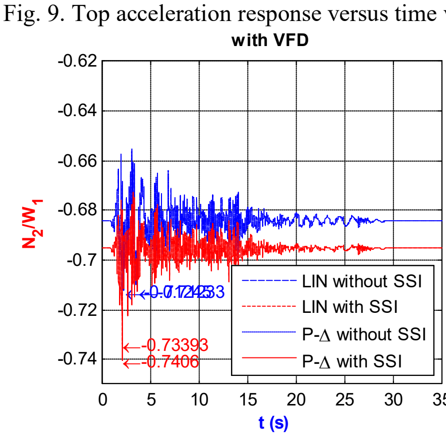


Fig. 11. Axial force of the 2nd column with FD

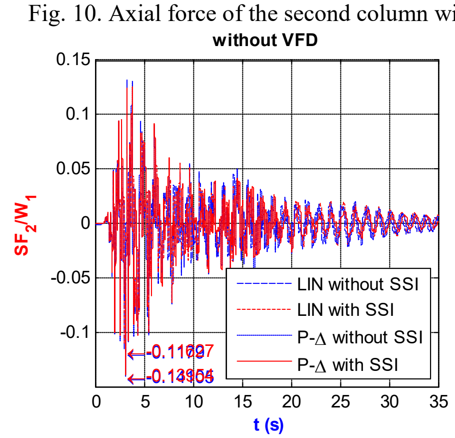


Fig. 12. Shear force at the end a of the 2nd column without FD

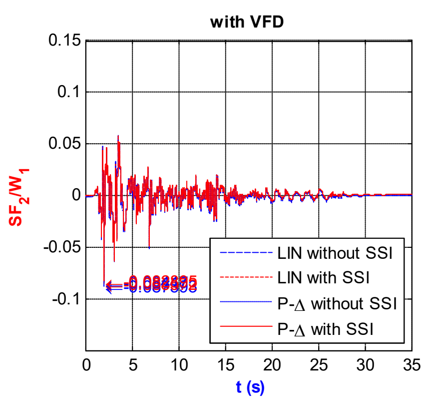


Fig. 13. Shear force at the end a of the second column with FD

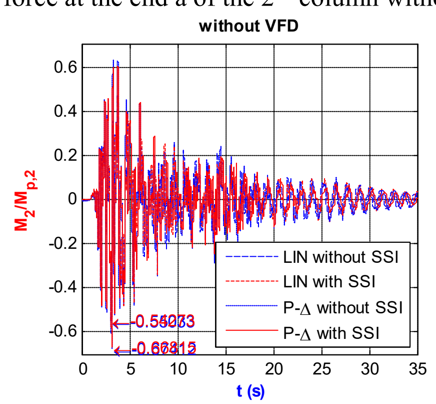
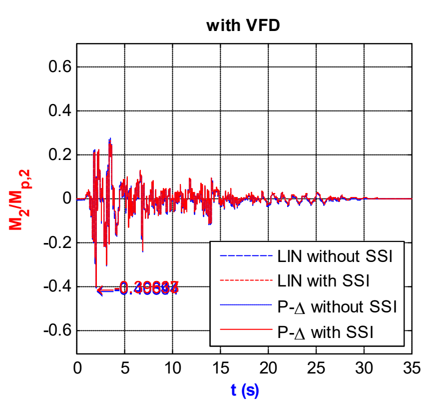


Fig. 14. Moment at the end a of the second column ($M_{p,2}=W_{p,2}.\,\sigma_y^{col}$) without FD



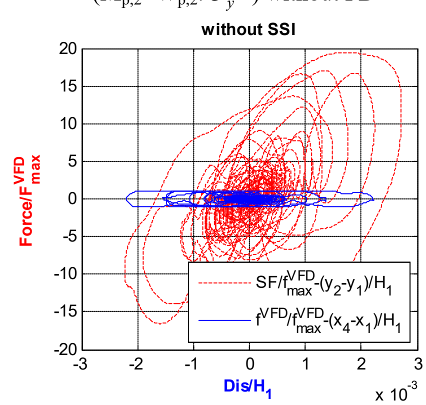


Fig. 15. Moment at the end a of the 2nd column with FD

Fig. 16. Hysteretic loop of Shear force and FD without SI

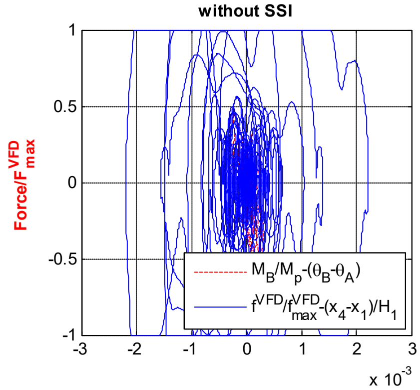


Fig.17. Hysteretic loop of Moment and FD without SI

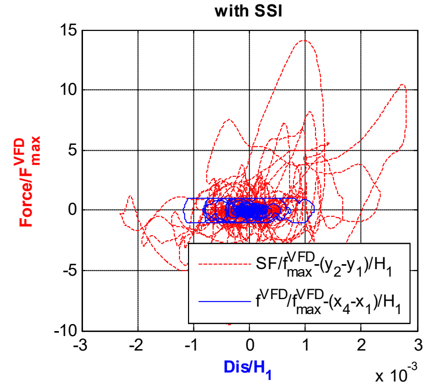


Fig. 18. Hysteretic loop of Shear force and FD with SI

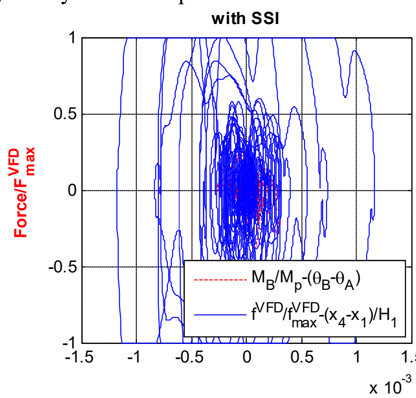


Fig. 19. Hysteretic loop of Moment and FD with SI

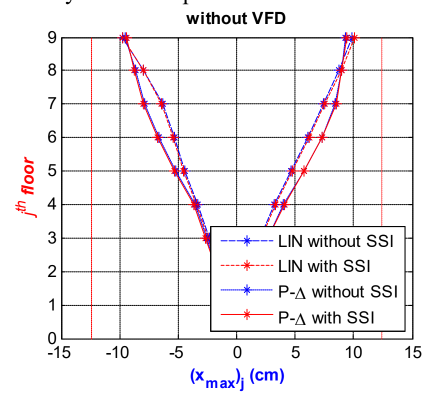


Fig. 20. Maximum story drift without FD

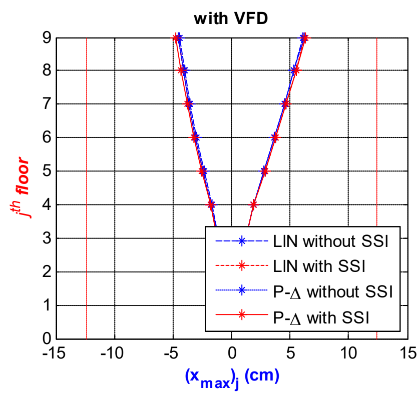


Fig. 21. Maximum story drift with FD

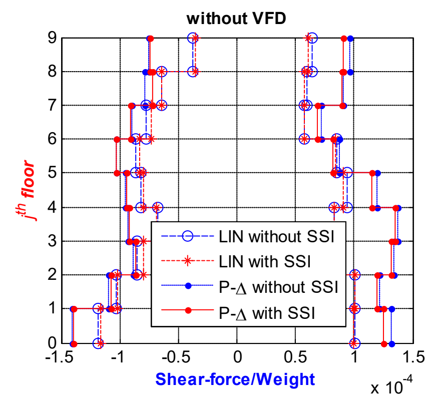


Fig. 22. Ratio of columns' maximum shear forces at 1-axis to its weight without FD

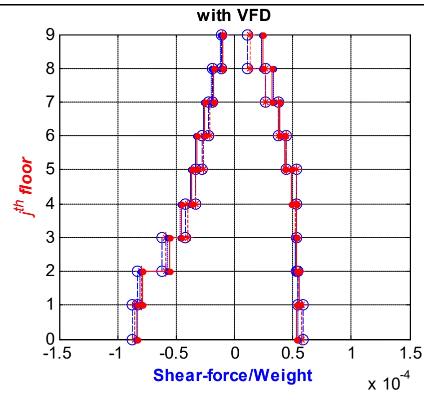


Fig. 23. Ratio of columns' maximum shear forces at 1-axis to its weight with FD

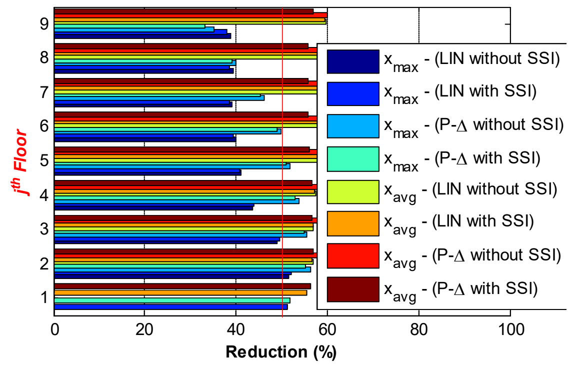


Fig. 24. Dynamic reduction in horizontal displacements and accelerations of the nine floors with and without FD

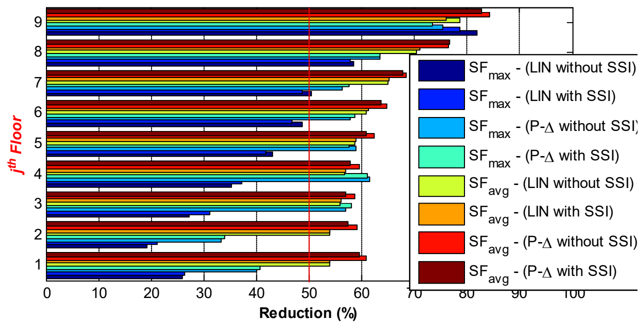


Fig. 25. Dynamic reduction in Shear force at column ends at the nine floors with and without FD

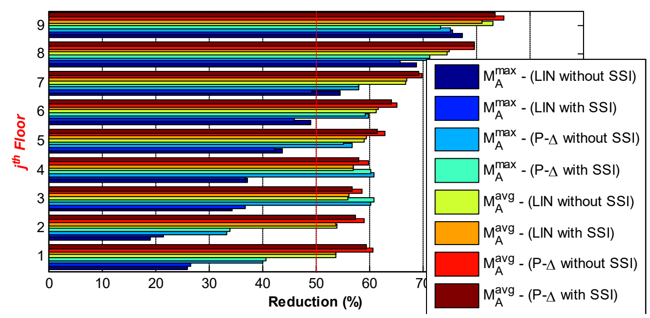
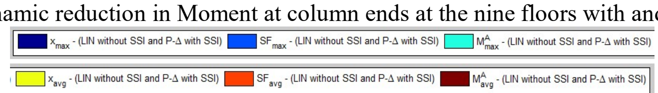


Fig. 26. Dynamic reduction in Moment at column ends at the nine floors with and without FD



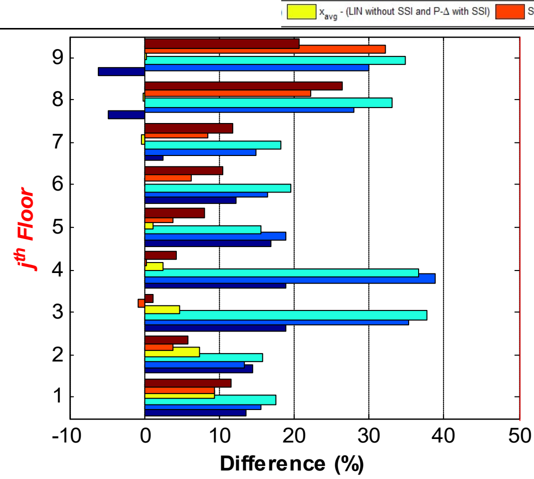


Fig. 27. Difference between P-Δ analysis and linear analysis of

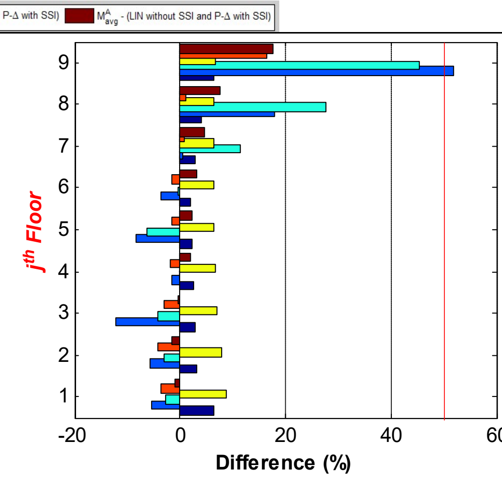


Fig. 28. Difference between P-Δ analysis and linear

the structure without FD	analysis of the structure with FD

Dynamic responses of the cases are compared in Fig. 6 to Fig. 28Fig. The simplest analysis (LIN without SI)$_{NCT}$ is not much different in maximum displacement response to other analyses consisting of (LIN with SI)$_{NCT}$, (P-Δ without SI)$_{NCT}$, and (P-Δ with SI)$_{NCT}$ (Fig. 6). However, it has a difference up to approximate 40% in internal forces including shear force and moment compared with (P-Δ with SI)$_{NCT}$ (Fig. 27). The acceleration responses of the four cases without FD are the same in large PGA (peak ground acceleration) zone but dissimilar in small PGA zone (Fig. 8). The axial forces of no-SI analysis are different to the axial force of SI analysis (Fig.10 and Fig. 11Fig). Equipped with FD, differences between linear and P-Δ analysis, non-SI, and SI are not considerable except 9th story (Fig. 28). The hysteretic loop of P-Δ analysis shows the nonlinearity of the relationship between force and displacement (Fig. 16 to Fig. 19). Area of the FD hysteretic loop is many times smaller than an area of the shear-force hysteretic loop (Fig. 18), but FD significantly contribute to dynamic responses of FD structures. FD reduces 60% average, 40% max displacement (Fig. 24), and up to 80% shear force and moment (Fig. 25 and Fig. 26). To enhance response reduction or a larger area of the hysteretic loop, FD could use higher maximum damper forces.

IV.

IV. CONCLUSION

This paper examines linear and P-Δ analysis of FD structures considering soil-structure interaction and subjected to seismic loading. In all cases of considering and not considering SI, linear, and P-Δ analysis, the nine-story FD structure expressions the acceptable dynamic reduction although it has different internal forces. In the cases of linear and P-Δ analysis, the paper illustrates a more accurate evaluation of dynamic responses caused by columns' large axial forces on the 1st floor. Additionally, in the cases of linear without SI and P-Δ with SI, the efficiency of FD is demonstrated in reducing dynamic responses. Linear and P-Δ analysis is different and acceptable provided that the value of FD damper force is sufficiently large.

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