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APPLICATIONS OF SEISMIC DAMPERS TO SPECIAL MOMENT FRAMES

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ABSTRACT

The application of seismic protection devices such as dampers has fundamentally altered the landscape of earthquake engineering and design. Structures designed and built without such devices typically use a code-prescribed design that implies structural damage, loss of operation, and possible replacement at design-level events. By contrast, seismic design incorporating earthquake protection devices reduce demand on structural and nonstructural members. Viscous dampers are robust, cost-effective, and have a proven exceptional performance record in past earthquakes. For buildings with viscous dampers, the initial cost of their utilization is at least in part neutralized by reduction in cost of other structural members. The long-term performance is the key parameter used for evaluation. A code-based design structure, could require major repair or replacement after a design level earthquake. In contrast, structures properly designed with viscous dampers will likely only require minimum post-earthquake inspection and limited damage. An example design is presented as an illustration.

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Introduction

Steel special moment frame (SMF)s are one of the common building systems in regions of high seismicity. The Northridge earthquake of 1994 demonstrated that the standard assumptions and construction detail (complete penetration welding of beam flanges to column flanges and bolted/welded shear tab) exhibited sudden and brittle failure. To address this issue, extensive testing and evaluations were conducted and prequalified connections have been developed [5] . Reduced beam section (RBS); see Figure 1, is a connection that is qualified for any size member. By reducing the beam flexural capacity, nonlinearity is concentrated in the reduced region and away from the potentially vulnerable beam-to-column connection.

The combination of viscous dampers and steel SMFs presents an attractive design option. The result is a highly damped, low-frequency building that limits seismic demand on structural and nonstructural components. Fluid viscous dampers (FVD)s are an ideal option due to their high damping because they are velocity dependent, and hence, do not significantly increase demand on foundations or columns. FVDs were originally developed for the defense and aerospace industries. The class of FVDs considered in this paper, is activated by the transfer of incompressible silicone fluids between chambers at opposite ends of the unit through orifices; see Figure 2. During seismic events, the devices become active and the seismic input energy is converted to heat and is thus dissipated.

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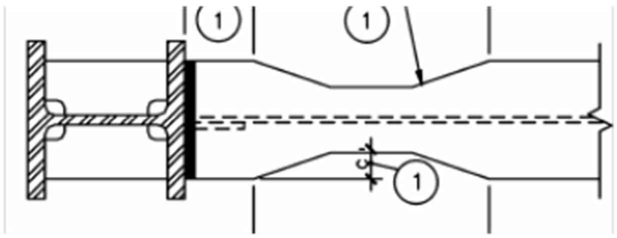
Application to steel SMF buildings

In the past several years, the authors have applied the design methodology discussed here for a number of steel SMF buildings. Sample structures are listed in Table 1. For a more detailed survey of other buildings with viscous dampers, see [10] .



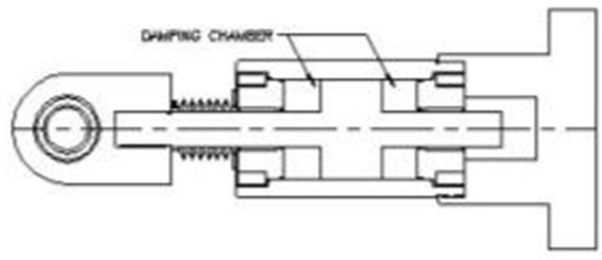


Figure 1. Details of RBS

Figure 2. Schematic of FVD [10]

Table 1. Sample of newly designed/constructed steel SMF with dampers

Structure	**Stories**	**Area, m^2**
Town Square	4	8,000
Sutter Gold, Modesto	5	13,000
CSU Sacramento AIRC Building	4	10,000
Vacaville Police Station	2	4,000
Ziggurat building	11	30,000

The additional cost of the dampers is at least partially offset by the savings in steel tonnage and foundation concrete volume. Hence, the conventionally designed and the damped buildings have similar initial costs. Sample data is presented in Table 2.

Table 2. Cost comparison for one of the buildings of Table 1

Item	**Conventional**	**Damped**	**Differential cost**
Moment Frames	274 Ton	223 Ton	- $150,000
Foundation	Reinforced concrete grade beams	No grade beams	- $200,000
Dampers	None	$200,000	+ $200,000
Net			-$150,000

US Case Study

US code provisions

Provisions of ASCE/SEI 7-10 [2] were used to design a new steel moment frame multi-story building in the Los Angeles area. The steel members were sized using conventional code design procedures [1] using reduced beam sections. FVDs were sized to control the story drifts. The dampers were placed only at the ground floor with pinned column bases where the maximum velocity is expected to occur. A parallel design was carried out using the conventional design methodology. This model was designed following the conventional code procedure for both strength and drift.

The four-story commercial building is 18.5 m tall and has a total floor space of 8,000 m^2. Architectural rendering of the building is presented in Figure 3. Computer program SAP [3] was used to prepare three-dimensional mathematical models of the damped and conventional designs.. For the damped model, the bases of all columns were modeled as pinned. For conventional design model, the fixity, provided by the grade beams, was assumed at the base of all columns. Figure 4 depicts the mathematical model of the building. Sixteen nonlinear FVDs were used to control story drifts at the first floor. The seismic mass of the building was approximately 9 MN.

Figure 3. Architectural rendering

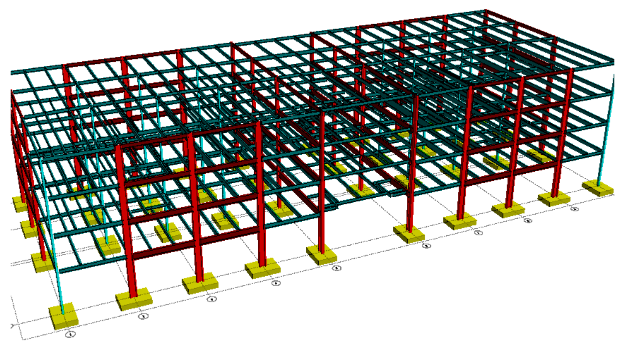


Figure 4. Mathematical model

Two levels of seismic hazard were investigated in design and include the maximum considered earthquake (MCE) with a 2,500-year recurrence interval, and the design basis earthquake (DBE) with a return period of 475 years, or 2/3 of MCE; whichever is greater. The response spectra for the two sites are shown in Figure 5. The peak DBE and MCE spectral accelerations were 1.4g and 2.1g, respectively. Spectrum-compatible records were synthesized using seeds from past earthquake records and having response spectra closely matching the target. The records have a typical duration of 40 seconds. Two performance levels were used in evaluation of building: life safety (LS) at DBE and collapse prevention (CP) at MCE.

Nonlinear response history analysis was performed to evaluate performance. The models were first preloaded with gravity load combinations and then subjected to the three pairs of accelerations at the DBE level and three pairs at the MCE level. The components of the ground motion were aligned with building principal axes. Maximum response quantities, such as, building floor displacement and accelerations, story shears, FVD forces, and member stresses, were extracted. The extreme values from all analyses were then used for evaluation.

Analysis results

The maximum computed story drift was approximately 1.4% for both damped and conventional structures, see Figure 6. Base fixity and larger member sizes control drift for the conventional model. FVDs provide such control for the damped model. The damped model has smaller base shear (Figure 7) and floor accelerations (Figure 8) because it has a larger period and damping ratio. Therefore, the application of the FVDs seismically protects both the structural and nonstructural components. In these figures, the system with convention code design is designated as CD, whereas, PBD denoted the model with viscous dampers.

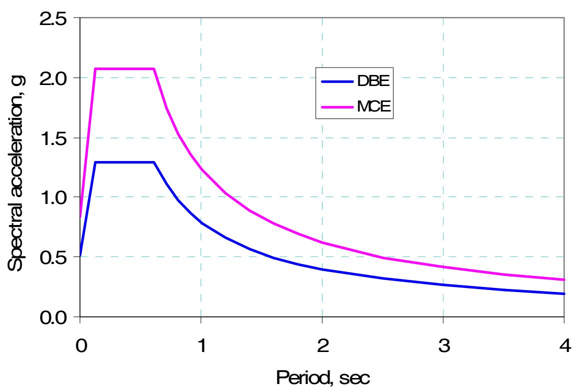


Figure 5. Seismic demand

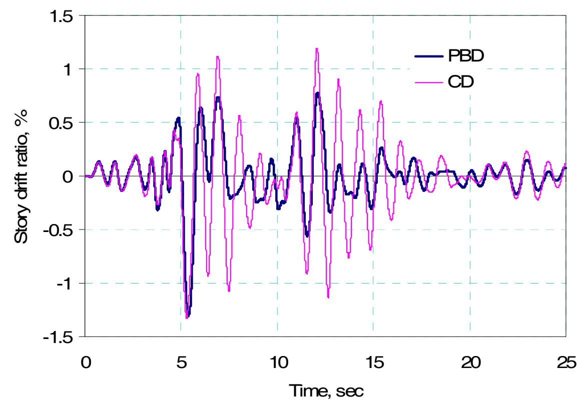


Figure 6. Story drifts

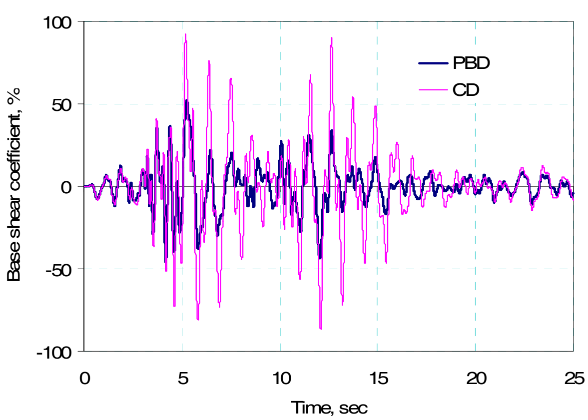


Figure 7. Base shear coefficients

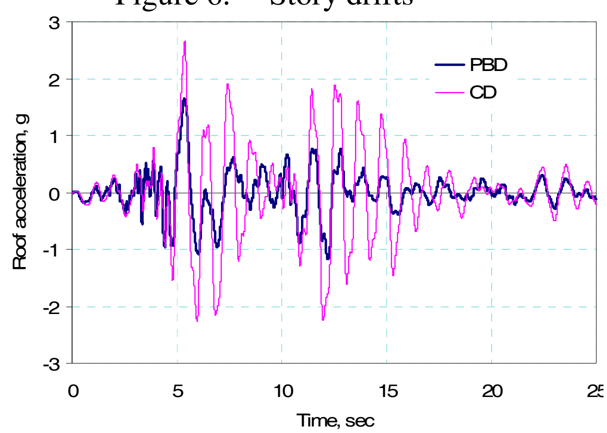
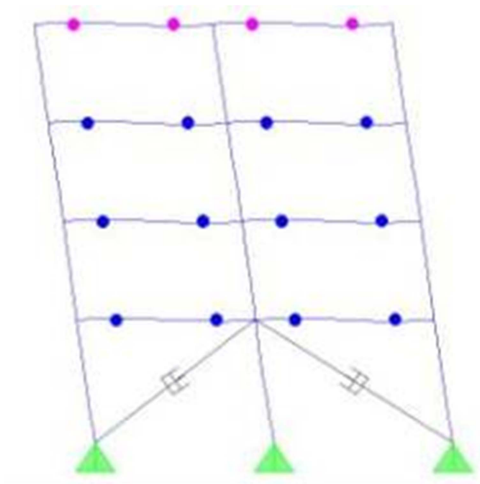
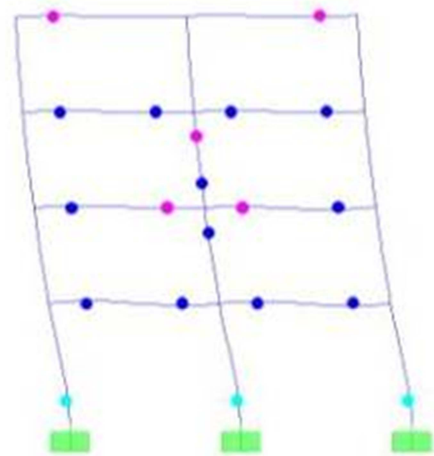


Figure 8. Roof accelerations

Figure 9 shows the snap shot of the PBD and CD models at maximum deformation for the MCE event. Both models meet their performance goal of collapse prevention for this event. However, the damped model meets the higher LS performance goal. Furthermore, as listed in Table 3, the plastic rotations are smaller for the damped model.



a. Model with dampers



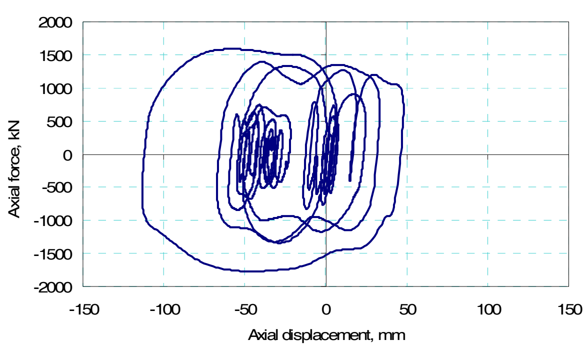
b. Conventional design

Figure 9. MCE plastic hinge rotations (Magenta, blue, and cyan denote nearly elastic, LS, and CP, respectively)

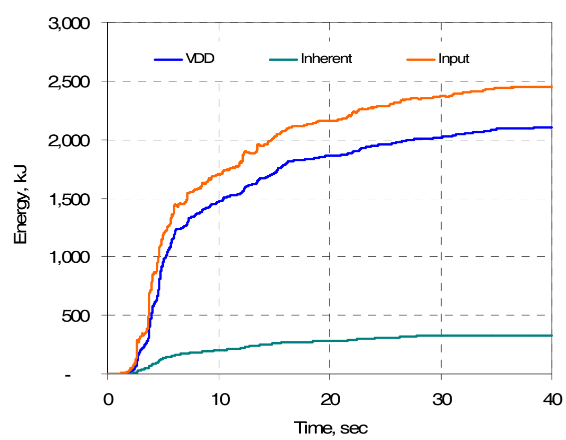
Table 3. Maximum MCE plastic hinge rotations, % radian

	Conventional	Damped
Beam	1.7	1.3
Column	2.6	--

The energy dissipation characteristic of the PBD model was analyzed further. Figure 10a presents the damper hysteresis loop and the components of seismic energy computed from analysis. In Figure 10b, the components on Input seismic energy are shown to be primarily dissipated by the viscous dampers (VDD) and a small amount due to the buildings inherent effective viscous damping.



a. FVD hysteretic behaviour



b. Components of seismic energy for elastic structure

Figure 10. Energy dissipated by dampers

Cost-Benefit Analysis

The PBD structure has superior long-term performance and the dampers do not require regular maintenance. Following a design earthquake, a properly designed conventional building would provide life safety, but could sustain significant damage because of yielding of ductile members

Viscous damper limit states

In most applications, FVDs are modeled as simple Maxwell model of Figure 11. The viscous damper itself is modeled as a dashpot in series with the elastic driver brace member. Such model is adequate for most design applications, but is not sufficiently refined for collapse evaluation. In particular, force and displacement limit states are unaccounted. Although dampers are comprised of many parts, the limit states are governed by a few elements. The dampers bottoms out, once the piston motion reaches its available stroke. This is the stroke limit and results in transition from viscous damper to a steel brace with stiffness equal to that of the cylinder wall. The force limit states in compression and tension are governed by the buckling capacity of the driver brace and the tensile capacity of the piston rod, respectively. Figure 12 presents the proposed limit state model for viscous dampers. This model is developed to incorporate the pertinent limit states and consists of five components: viscous element, drive brace stiffness, FVD piston stiffness, FVD cylinder in tension, and FVD cylinder in compression.

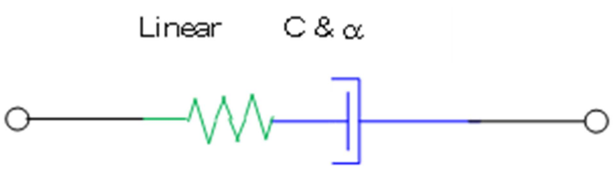


Figure 11. Maxwell model

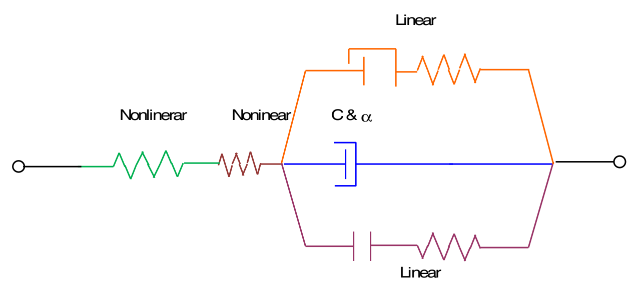


Figure 12. Limit state model

For analysis, once the stroke limit is reached, the damper becomes numerically equivalent to a steel brace. Upon unloading, the damper reverts back to a FVD. When the force limit is reached, the entire damper is ineffective and thus permanently removed, even after unloading. The sudden transmissions between viscous damper, steel brace, and no members can impart large impact forces on the structure. At the instant that the gap closes, the damper force is zero. However, as loading is continued, the unit displacement can increase due to deformation in the cylinder wall and thus velocity is non-zero. At the large peaks, the damper force, which is algebraic sum of the force in the dashpot and the cylinder wall, can be smaller than the force resisted by the wall cylinders.

Analysis of damped structures subject to large earthquakes

Background

The input histories used in analysis were based on the two components of the 22 far-field NGA PEER [9] records. These 44 records have been identified by FEMA P695 [7] for collapse evaluation analysis. The selected 22 records correspond to a relatively large sample of strong recorded motions that are consistent with the code [1] and are structure-type and site-hazard independent.. For analysis, the records were normalized to remove the record-to-record variation in intensity.

Program OpenSees [8] was used to conduct the nonlinear incremental dynamic analyses or IDA [11] described in this paper. Pertinent model properties are listed here. To illustrate the concepts described in this paper, design and analysis of a of 5-story archetypes (see Figure 13) with viscous damping was conducted. The basic geometry and distribution of dampers for these models are summarized in Table 4.

Table 4. Archetypes

Archetype	Stories	Column base	Target Drift Ratio	FVD Safety Factor
B1	5	Fixed	2.0%	1.0
B2	5	Fixed	1.0%	1.3

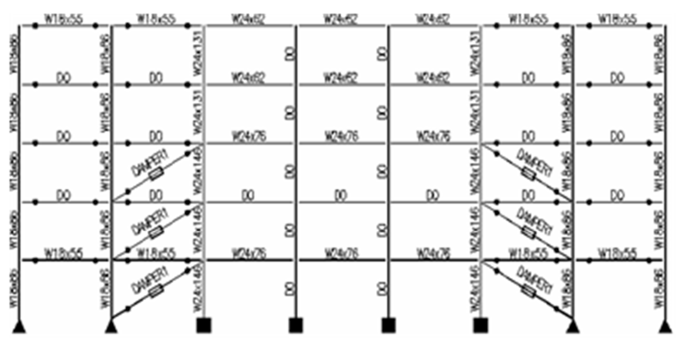
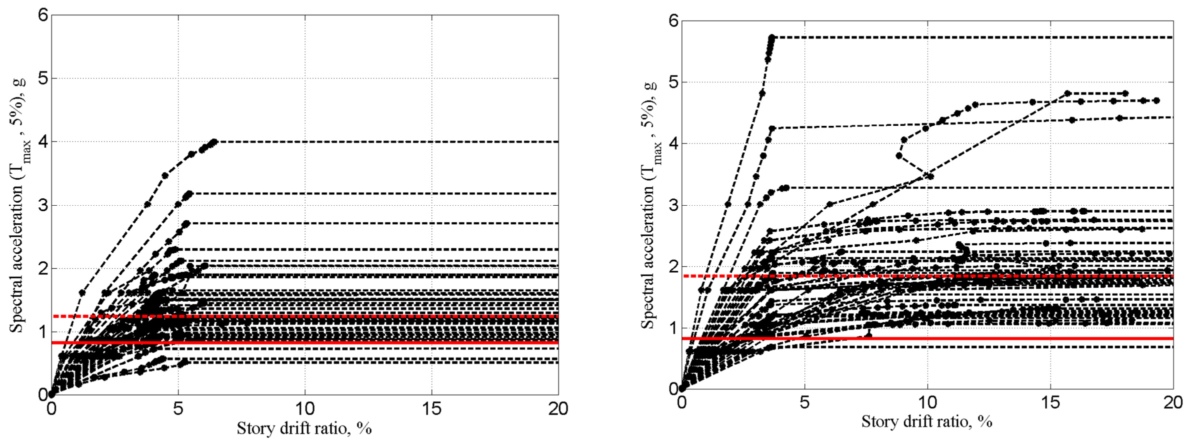


Figure 13. Five-story archetype B1 (Bases of columns of SMF are fixed and those of gravity columns are pinned)

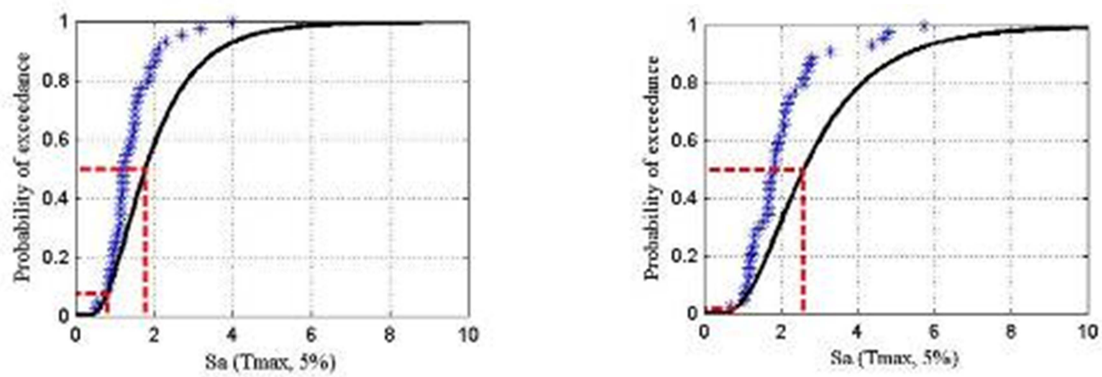
Analysis results

The analysis results of the five-story archetype are presented in Figure 14. The computed system ductility was 8.0 and resulted in a spectral factor (SSF) of 1.34. For the IDA plots, the solid and dashed red lines correspond to the MCE (SMT) and the median collapse capacity (SCT), respectively. Note that the addition of small FVD factor of safety significantly increases collapse margin. For the fragility plots, the 44 collapse data are statiscally organized and a lognormal curve is fitted to the data (dashed lines in the Figure 14). The plot was then rotated to correspond to a total uncertainty of 0.55 (solid line) per FEMA P695. Finally the curve was shifted to account for the effect of the SSF (dark solid lines in the Figure 14). The probability of collapse at MCE intensity was then be computed. The probability of collapse at MCE level was reduced by a factor of approximately 4 when an additional damper factor of safety of 30% is included. Table 5 summarizes the results. The collapse margin ratio (CMR) is defined as the ratio of SCT and SMT. The adjusted collapse margin ratio (ACMR) is then computed as the product of SSF and CMR. FEMA P695 specifies a minimum ACMR of 1.59 for acceptable performance. Both archetypes have significantly larger collapse margins and therefore pass.



B1 B2

IDA curves



B1 B2

Fragility plots

Figure 14. Analysis results

Table 5. Damper fragility data

							Response probability at MCE	
Archetype	**SCT**	**SMT**	**CMR**	**SSF**	**ACMR**	**P/F**	**Collapse**	**Damper capacity**
B1	1.24	0.82	1.51	1.34	2.20	Pass	8.0%	22%
B2	1.81	0.82	2.25	1.34	3.10	Pass	2.0%	10%

Conclusions

New steel buildings were designed using performance based engineering (PBE) and provisions of ASCE 7. SMRFs were used to provide strength; dampers were used to control story drifts. PBE design using dampers is superior to the conventional design. The demand on both structural and nonstructural components is reduced. To date, a model of viscous dampers with limit states has been formulated that includes damper limit states. Current research using IDA and limit states of dampers provides a more realistic assessment of the performance of moment frames with dampers. All the archetypes had significant margin against collapse and thus had satisfactory performance. When a damper factor of safety is included in design, additional protection for the structures and dampers is provided. As one of the research deliverables, pertinent information will be provided for the designers to assist in seismic design using this approach

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