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DAM as a primary method of structural steel design for stability

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***Abstract*— *Transitioning from a method that does not account for all types of loads, geometric imperfections, and construction flow to a reliable method; the best way is to evaluate their feasibility by design practice. This paper displays a comparative analysis of Effective Length Method (ELM) and Direct Analysis Method (DAM) as the main approaches to stability analysis and design of steel structures. Afterward, the paper detailed the DAM application with second-order analysis to account for P-Δ and P-δ effects, notional load for geometric imperfection, and flexural and axial stiffness reduction to account for inelastic effects*. Verification on a 10-story building illustrated DAM to be more comfortable, faster, and ensuring that second-order effects are entirely performed. As the ELM is limited to Δ2nd order/Δ1st order < 1.5, the design analysis check results illustrate that second-order analysis in the DAM leads to a feasibility analysis of ELM.

***Keywords*— Direct Analysis Method (DAM), Design for stability, Effective Length Method (ELM), Notional loads, Second-order analysis.**

I. INTRODUCTION

Stability shall be provided to a structure as a whole and each of its elements. The effects of all the following on the structural stability and its elements shall be considered: (1) flexural, shear and axial member deformations, with all other distortions that contribute to the displacement of a structure; (2) second-order effects (both P-Δ and P-δ effects); (3) geometric imperfections; (4) stiffness reductions due to inelasticity; and (5) uncertainty in stiffness and strength. All load-dependent effects shall be calculated at a level of loading corresponding to LRFD load combinations or 1.6 times ASD load combinations. This paper focuses on the significance of considering second-order effects in structure analysis.

Conducting this study, two preferred methods of structural design for stability: direct analysis method (DAM) and effective length method (ELM) are compared to distinguish one’s competitiveness. DAM was found competitive in the various studies as the best method that accounts for both the structural element and the overall frame system stability (M. A. PaL, P. W. Sauer, K. D. Demaree). As it is shown in the results of this paper, in most scenarios, both ELM and DAM show consistent results that adequately capture significant characteristics that control the behavior of a steel frame. This paper proposes that an evaluation of cases in which each one of these methods may not be appropriate has to be conducted. One way to evaluate the feasibility of both methods is simplifying the DAM to second-order analysis method along with the application of notional loads. Section 2 provides the reader with details about ELM and DAM; section 3 discusses the satisfaction of second-order effects; The DAM application detailed in section 5; section 6 comprises the application of the DAM, and section 7 concludes the study.

II. METHODS OF STRUCTURAL DESIGN FOR STABILITY

2.1 Effective length method

Codes of practice rely on an effective length method to assess the stability of multistory frames. The ELM involves isolating a critical column within a frame and evaluating the rotational and translational stiffness of its end restraints, so that the critical buckling load may be obtained. It allows the buckling capacity of a member in a structural system to be calculated by considering an equivalent pin ended column in Euler buckling.

The ELM accounts for the influence of the total frame on the behavior of an individual column. As expected, in many cases, the frame to be analyzed does not comply

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with all the assumptions listed in the AISC specification regarding ELM, yet designers still use this method to design frame structures. A significant flaw with this method is that for many frame structures to 'fail,' several members within the structure need to fail first. Therefore, the load-carrying capacity for the entire frame is overestimated. In many situations, it makes the ELM too conservative. Thus, tends to counterbalance the effect of inconsistencies between frame behavior and the assumptions that form the basis for this method. Another drawback of this method is its inability to account for the effects of imperfections, which include the out-of-plumbness of the frame.

2.2 Direct analysis method (DAM)

Because the ELM is based on several assumptions on geometry, boundary condition, and material properties of columns, it may not always be appropriate for the design of steel columns, especially for frame configurations in which the conditions of a given column are not consistent with these assumptions. In an attempt to provide a design methodology that would more accurately capture the main factors that affect column behavior, the Direct Analysis Method DAM was introduced into the steel design specifications in 2005 as an alternative to the more traditional ELM [1] [2]. It was upgraded to Chapter C in the 2010 Specification as the primary method to design structures for stability. A significant advantage of the DAM is its ability to account for member and construction imperfections within a frame, which creates additional stresses and reduces the load-carrying capacity of the structure. Therefore, the features of the DAM include P-Δ and P-δ effects, which are accounted for directly through second-order analysis. Geometric imperfections are accounted for, through direct inclusion in the analysis model, or by applying notional loads to displace the structure. Inelastic effects such as distributed plasticity are accounted for in analysis for using flexural and axial stiffness reductions; the best part is that we design using K=1 no more factors.

2.3 The transition from ELM to DAM

Both methods use column interaction equations to estimate the capacity of individual steel columns. The fact that ELM and DAM are different, they will not necessarily produce the same column sizes for a given structural configuration. If the two methods produce substantial differences in size, the adequacy of each method to provide structural members with sufficient capacity to resist the imposed loads becomes a concern. For the many engineers transitioning from the ELM to the DAM, the best way is to learn by practical scenarios. This paper details the design of a 10storey office building to draw out a practical computational example using DAM.

DAM for the design of steel structures is recommended in significant design codes, including EN1993-1-1 [1], CoPHK [2], and ANSI/AISC [3], and has been proven as a reliable method by several researchers. It is a quasi-simulation-based method which employs the Finite-Element (FE) method to capture the actual behaviors of members and structures directly. In the analysis, the significant effects relating to the stability, such as the equilibriums on the deformed shapes, the residual stresses, the initial member out-of-straightness, and the global imperfections, should be comprehensively considered. Therefore, a robust numerical analysis method, being capable of considering these factors, is crucial. [4]

2.4 Comparison between the direct analysis method (dam) and effective length method (elm):

In ELM, notional loads are minimums and negligible, but when the AISC 2010withame out now, ELM requires the use of Notional load according to the specifications. For analysis using ELM, notional is 0.002*gravity loads, and they must be used in combination with the gravity load. Noting that notional load cannot be added to other lateral loads; they are minimums. In case gravity loads combined with notional loads will probably only control of structures with high gravity loads and low lateral loads.

DAM requires explicit consideration of initial geometry imperfections, which is mostly done by applying notional loads to displace the structure. The notional load can be applied as a minimum lateral load acting concurrently with the gravity load for structure second-order effects ≤ 1.7; the for second-order greater than 1.7, the notional load must be added to the lateral load and acting in the same direction with the lateral load.

Table 1: Comparison between DAM and ELM

	Effective Length Method (ELM)	**Direct Analysis Method (DAM)**
Type of Analysis	Second-order or Amplified First Order	Second-order or Amplified First Order
Member stiffness	Nominal EI & EA	Reduced EI & EA
Notional loads	0.002Yi Minimum	0.002Yi Minimum if $\Delta_{2nd\ order}/\Delta_{1st\ order} \leq 1.7$ Additive if $\Delta_{2nd\ order}/\Delta_{1st\ order} > 1.7$
Column effective	Side-sway buckling	K=1

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length	analysis-determine K	
Limitation	$\Delta_{2nd\ order}/\Delta_{1st\ order} \leq 1.5$	No limitation
AISC 360-10, 16 Specifications	Appendix 7	Chapter C

III. SECOND-ORDER EFFECTS

It results from displacement from the structural frame and also from the curvature of the individual members. Equilibrium must be satisfied with deformed geometry of the structure; this includes both system-level effects "P-Δ effect" and member level effects "P-δ effect; In other stability design method b1 and b2 account for these effects. In the DAM, the second-order effects are directly modeled. The P-δ effect is the member level effect, as mentioned, equilibrium must be satisfied with deformed geometry. And the member curvature produces additional moment in the member

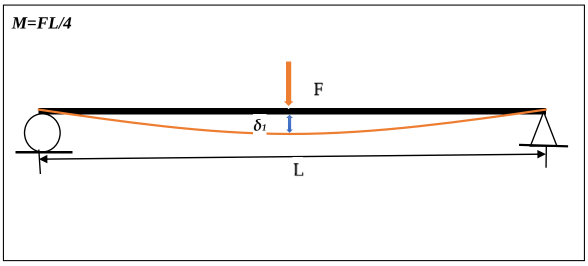

Fig.1: P-δ effect

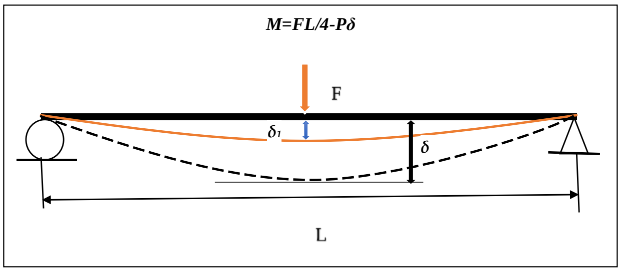

Fig.2: P-Δ effect P-δ effect

When applied to an axial compressive load (P), an additional deflection and momentum result from the axial load acting on the curve moment geometry. Therefore, the total midspan deflection = δ and the total mid-span momentum = $M=FL/4-P\delta$. The $P\delta$ component is the second-order effect. *P-Δ effect:* this is an overall system-level effect, again equilibrium must be satisfied on the deformed frame geometry. Gravity displacement produces thrust on the system.

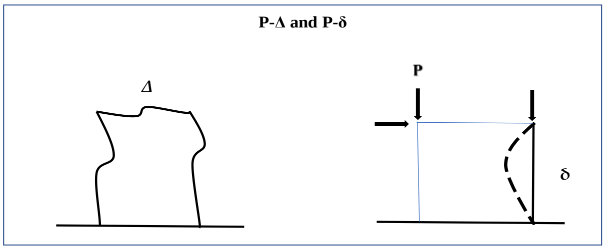

Fig.3: second-order effects

IV. DIRECT ANALYSIS METHOD APPLICATION

4.1 Accurate model frame behavior

It is necessary to accurately model the structure system in order to capture the correct behavior accurately. Thus, it includes correcting: Model geometry, member size orientation, boundary conditions that accurately reflect actual conditions, member properties of stiffness, and incomplete loads, including effects in leaning columns. All seems pretty basic, but the model must represent the actual structural behavior (Fig.4).

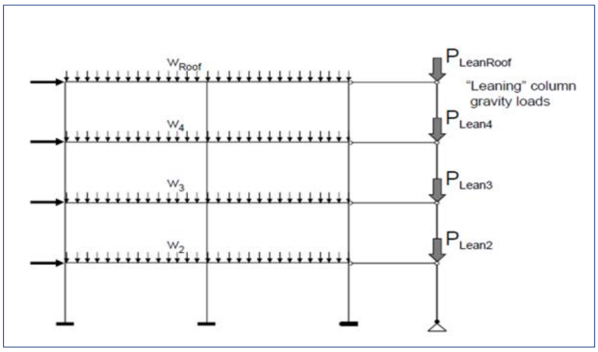

Fig.4: Leaning column effects added to the structure model

By leaning columns, it means that gravity columns supporting gravity loads but not parallel lateral load resisting system. Gravity columns lean on the lateral system for stability. It is something missed a lot when analyzing a frame with an isolated 2 D model. Forgetting to add effects of all gravity columns that rely on that frame for support destabilize the frame.

The right column (Fig.4) representing all the gravity columns stabilized by the frame analyzing. As the frame deflects, this column leans on the lateral system for support, and this gravity column also deflects and adds to the overall P-Δ effects. P-Δ effects from this leaning column work to destabilize the frame, so they must be included in the analysis. All loads and conditions that work to destabilize the structure system should be included in the structural model

4.2 Factor all loads (LRFD and ASD)

Loads have to be factored. Even for Allowable Strength Design (ASD) because, when trying to capture the structural behavior of strength limit state, it is remarkable that nonlinear structural behavior can reduce second-order effects as we push to strength limit state. Furthermore, using LRFD, we must factor our loads up to the factored strength limit level. ASD factors all loads by 1.6 then divide the resulting forces by 1.6 again; this accurately captures the structural behavior of the strength limit state. Factoring all loads, include all loads that affect stability, including leaning columns and all other destabilizing loads.

4.3 Consider initial imperfection:

Consider initial frame imperfections, usually by the application of notional loads. It is well clear that all buildings cannot be built perfectly. Initial imperfections such as (1) member out-of-straightness "P-δ effects," (2) story out-of-plumbness "P-Δ effects" resulted from fabrication and rational tolerances increase destabilizing effects that destabilize the structure. Only the initial out-of-straightness (δ_o) effects contribute to the column curves. In the DAM, the story out-of-plumbness (Δ_o) needs to be accounted for an analysis model; this can be done by modeling an out-of-plumbness geometry or by applying a notional load to laterally deflected structure.

The notional load is applied to displace the structure laterally to reduce an initial out-of-plumbness. This notional load needs to be implied at each framing level based on the distribution of gravity loads. Specifications define notional lateral load equal to 0.002*gravity load applied at that level. Notional loads should be applied simultaneously with gravity loads at each level in the direction that most stabilizes the structure.

The 0.002 factor consists of the code of standard practice tolerance HR500 for column or out-of-plumbness. If a little out-of-plumbness tolerance is specified, and assure that the little tolerance will be achieved. Thus, it can reduce the 0.002 factor accordingly to little tolerance. "if relaxed tolerances are allowed or perhaps it is necessary to check an out-of-tolerance condition, then a higher factor should be used based on a larger out-of-plumbness condition.

Initial geometric imperfections are considered by applying "notional loads" or "notional displacements" The specification defines Notional Loads as Ni = 0.002αYi, where α = 1.0 (LRFD), 1.6 (ASD) to make sure that notional loads are factored as the gravity loads. Yi is a gravity load applied at level i. The essence here is that notional loads are factored loads. Ni is added to other loads to apply in the same direction as the other lateral loads (if the wind load is acting to the right, the notional load acts likewise, and so on). Notional loads are likely to be applied to the direction that requires the most destabilizing effects. Specifications do not have a specific law If $\Delta_{2nd\ order}/\ \Delta_{1st\ order} < 1.7$ (reduced stiffness), or, if $\Delta_{2nd\ order}/\ \Delta_{1st\ order} < 1.5$ (nominal stiffness), then permissible to omit Ni in combinations with other lateral loads.

4.4 Reduced stiffness that contributes to the stability

The consequence of differential cooling rates during manufacturing. Results in earlier initiation of yielding (some sections yield before other sections), thus affecting compressive strength; Lowers member flexural strength and buckling resistance. the stiffness of all members that contribute to stability is reduced, To account for the facts of residual stresses of distributed plasticity. For axial stiffness: (EA* = 0.8EA) and Flexural stiffness (EI* = 0.8 τ_b EI, $\tau_b \le 1.0$); depending on the magnitude of the Axial load. For lower Axial load when the required strength is lesser than the usual load: $\tau_b = 1.0$ when $\alpha Pr/Py \le 0.5$.

For high Axial load, τ_b is less than one and is calculated based on the ratio of the required Axial strength to the usual strength, and the resulted flexural stiffness becomes less than 0.8EI. τ_b =1 when the actual load is lesser than *Py*. τ_b =1 often wanted for moment frames. The specification offers a simple τ_b Simplification that allows the increase of 0.001αYi to notional loads (Ni) so (Ni=0.003αYi instead of 0.002αYi) then $\tau_b = 1$

- τ_b: $\tau_b = 1.0$ when $\alpha Pr/Py \le 0.5$
- $\tau_b = 4(\alpha Pr/Py)[1-(\alpha Pr/Py)]$ when $\alpha Pr/Py > 0.5$
- α = 1.0 (LRFD), 1.6 (ASD)
- (τ_b simplification: τ_b = 1.0 can be used if 0.001αYi added to Ni), (Ni = 0.003αYi instead of 0.002αYi)

4.5 2nd-order analysis

It is necessary to know how a software address P-Δ and P-δ. For more software packages, it is necessary to mesh the compression element into smaller segments in order to capture P-δ effects accurately. The number of segments to mesh depends on several factors, including how the software handles the P-δ and magnitude of secondary effects.

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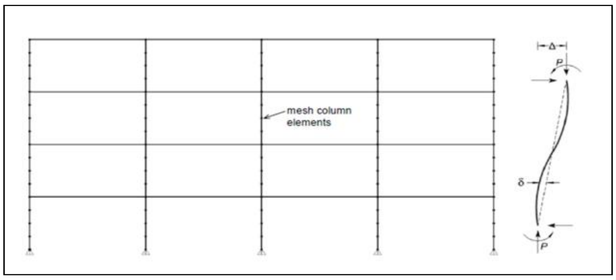

Fig.5: Internally mesh compression elements to capture P-δ effects

(Fig.5) mesh the compression element into four segments. If there is no compression in beams such as in a moment frame like in (Fig.5), there is no need to mesh then since there will not exist significant P-δ effects in those beams. If there exist compression beams, it is necessary to mesh them as well. Some software packages enable to mesh of the frame elements adequately. (Ex: SAP 2000) Make sure about how the software handles secondary analysis in stiffness reductions.

For SAP2000, Reduction factors to EI and EA are assigned after running the design check. Iteration needs to be done as changing member sizes or loads; it is necessary to reduce the cycle of running the analysis performing the design check, then rerun the analysis that calculates the stiffness reductions and then perform the design check again. $\Delta_{2nd\ order}/\ \Delta_{1st\ order}$ ratio has to be checked After the member sizing:

- If $\Delta_{2nd\ order}\ /\ \Delta_{1st\ order} \leq 1.7$ (reduced stiff.) or 1.5 (nominal stiff.), then *Ni* not required in lateral combinations (*Ni* only *required* in gravity combinations)
- If $\Delta_{2nd\ order}\ /\ \Delta_{1st\ order} > 1.7$ (reduced stiff.) or 1.5 (nominal stiff.), then include *Ni* in all load combinations.
- Simplification: include *Ni* in all load combinations, then no need to check $\Delta_{2nd\ order}/\Delta_{1st\ order}$ ratio

4.6 K factor and member design

The DA method accounts for both P-Δ and P-δ, and Geometric imperfections considered explicitly. Therefore, no more *K*-factors because K-factor since it is always misapplied. Loss of stiffness under high compression loads will be accounted for during analysis by reducing member stiffness. The net effect amplifies second-order forces to comes close to the actual response. For allowable strength design (ASD), resulting analysis forces should be divided by 1.6 since they were factored by 1.6 for analysis. Keeping in mind that, Since the Analysis is not linear if any member size or load changes, it is necessary to rerun the analysis and recheck the designs

4.7 Reduced stiffness for serviceability checks

Reduced stiffness is only used in strength analysis. Whereas, unreduced stiffness is used for serviceability checks: vibration and drift limits for wind and seismic are checked using nominal (unreduced) stiffness, and building periods are determined using nominal (unreduced) stiffness.

V. STRUCTURAL DESIGN FOR STABILITY IN SAAP

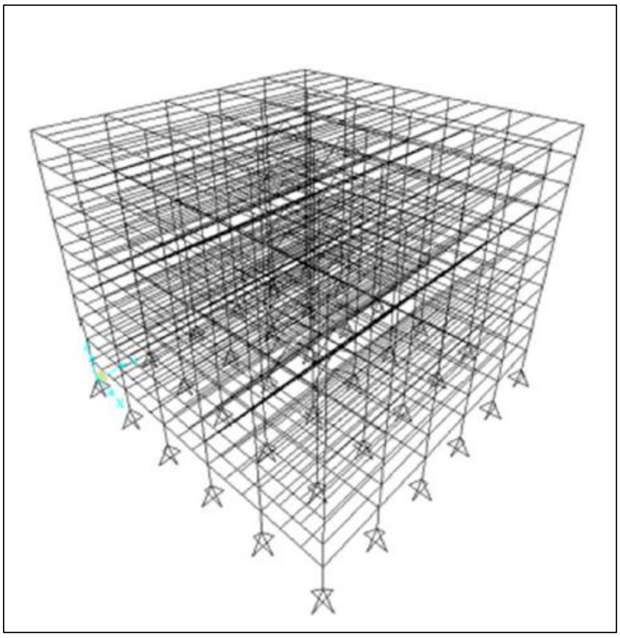

Fig.6: Accurate model frame behavior

4.8 Gravity Loads

Floor:

- Composite steel deck (3" +3$_{1/2}$" slab, LWC) = 50 psf
- Superimposed dead load + floor framing = 15psf
- Wall load = 25 psf (over floor area at all levels)
- Live Load = 100psf (reducible)

Roof:

- Same dead loads as floor
- Live Load = 30psf (unreduced)

1.1 Live Load reduction

Applied according to section 1607.10, IBC 2012

$$L = L_0\left(0.25 + \frac{15}{\sqrt{k_{LL}A_T}}\right) \quad \textbf{(1)}$$

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K_{LL} = Live load element factor: 4 for columns-interior, exterior w/o cantilever slabs, 2 for beams–interior, edge w/o cantilever slabs. For beams of moment frames (Table 2)

$$L = 100\left(0.25 + \frac{15}{\sqrt{2*15*30}}\right) \quad \textbf{(2)}$$

1.2 ASCE 7-05 wind loads

- Basic wind speed, V = 90 mph
- Exposure Type B
- Occupancy Category = II
- Importance Factor, I = 1.0
- Wind directionality factor, K_d = 0.85
- Topographic factor, K_{zt} = 1.0
- Gust effect factor, G = 0.85

1.3 Auto-generation option utilized in SAP

- ASCE 7-05 seismic loads
- S_s = 0.317g; S_1 = 0.106g
- Site Class D
- Occupancy Category II
- Importance Factor, I = 1.0
- S_{DS} = 0.327 g; S_{D1} = 0.168 g
- SDC = C
- Steel Systems Not Specifically Detailed for Seismic
- Resistance - R = 3; Cd = 3
- Equivalent Lateral Force Procedure
- Approximate fundamental period: $T_a = C_t h_n^x$ = 125 ft
- For moment frame direction, C_t = 0.028, x = 0.8
- For braced frame direction, C_t = 0.02, x = 0.75
- For S_{D1} = 0.168 g, C_u = 1.564

Upper limit on period

- T = 2.08 sec for moment frame
- T = 1.17 sec for braced frame

1.4 Notional load

- Yi (Dead) = 65 psf + 25 psf + 10 psf (partitions) + 10 psf = (vertical framing) = 110 psf
- Yi (Floor Live) = 100 psf
- Yi (Roof Live) = 30 psf
- NDead = 0.002 x 110 psf x 150 ft x 150 ft = 5 kips
- NLive = 0.002 x 100 x 150 x 150 = 4.5 kips
- NLive R = 0.002 x 30 x 150 x 150 = 1.4 kips

Noting that Torsional cases should also be considered. For coupled or correlated systems, Nx & Ny should be applied simultaneously with appropriate directional correlation. (Table 4)

VI. ANALYSIS

1.5 Strength Design Analysis

A second-order elastic analysis is performed, including P-Δ and P-δ effects using reduced member properties. Property modifiers for the analysis: EA*= 0.8EA, and EI* = 0.8 τ_bEI. Assuming that τ_b = 1.0.

1.6 Serviceability Analysis

For serviceability checks, the second-order elastic analysis is performed, including P-Δ and P-δ effects using the nominal member properties. (

Table **5**, Table 6,

Table *7*)

✓ From ASCE 7-05 Table 12.12-1, allowable story drift = 0.020hsx = 0.020 x 150 in. = 3 in.

✓ Max. story drift = 0.79” (level 9)

✓ Inelastic drift = 3 x 0.79” = 2.37 in. < 3 in → OK

Table 2: Live load reduction

Levels	Interior Column			With 100psf design LL			With 75 psf LL			Correction in Load
	Kll = 4			P live kips	P live * LLR kips	P live * LLR kips	P live kips	P live kips	P Up live kips	P Upper level (kips) for column LLR
	Tributary area of reduced load									
	SF	SF	LLR							
Roof	0	0	1	0	0	0	0	0	0	0
Level 10	900	900	0.5	90	90	45	67.5	67.5	22.5	22.5
Level 9	900	1800	0.4	90	180	70.6	67.5	135	58.2	35.7
Level 8	900	2700	0.4	90	270	108	67.5	203	94.5	36.3
Level 7	900	3600	0.4	90	360	144	67.5	270	126	31.5
Level 6	900	4500	0.4	90	450	180	67.5	338	158	31.5

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Level 5	900	5400	0.4	90	540	216	67.5	405	189	31.5
Level 4	900	6300	0.4	90	630	252	67.5	475	221	31.5
Level 3	900	7200	0.4	90	720	288	67.5	540	252	31.5
Level 2	900	8100	0.4	90	810	324	67.5	608	284	31.5

Table 3: Column load design

Column Lebel: B-2					**Area service load**				**Cumulative factored load**						**Column**	
	FI. Label	FI. Height (ft)	Fy of col	KLL	Load type No.	Trib. Area (ft*2)	Load type No.	Trib. Area (ft*2)	Dead Load (kips)	S-Dead Load (kips)	reducible live load (kips)	Unreducible live Load	Total load (kips)	Column size	Col. Cap (kips)	Pu/Pn
No															Col. Designer	
10	Roof	12.5	50	4	3	900	2	900	81	16.2	0	43.2	140.4	W14x30	189.8	).74
9	10	12.5	50	4	1	900	2	900	163.1	32.4	72	43.2	310.7	W14x43	357.7	).868
8	9	12.5	50	4	1	900	2	900	245	48.6	122.9	43.2	459.7	W14x61	612.4	).751
7	8	12.5	50	4	1	900	2	900	327	64.8	172.8	43.2	607.8	W14x68	685.8	).888
6	7	12.5	50	4	1	900	2	900	409.3	61	230.4	43.2	763.9	W14x82	628.5	).824
5	6	12.5	50	4	1	900	2	900	491.6	97.2	288	43.2	920	W14x90	1057.7	).87
4	5	12.5	50	4	1	900	2	900	574.1	113.4	345.6	43.2	1076.3	W14x99	1162	).926
3	4	12.5	50	4	1	900	2	900	656.9	129.6	403.2	43.2	1232.9	W14x120	1412.2	).873
2	3	12.5	50	4	1	900	2	900	739.9	145.8	460.8	43.2	1369.7	W14x132	1554.2	).894
1	2	12.5	50	4	1	900	2	900	823.1	162	518.4	43.2	1546.7	W14x145	1732	).893
													1548.6			

4.9 Design analysis check

About the result in

Table ***8***, Δ2nd order/Δ1st order ≤ 1.5 (nominal properties), the design analysis is OK; notional lateral loads are only required with gravity loads. Comparing the Design with ELM: Using the DAM, the drift controlled moment frame had Δ2nd order/Δ1st order < 1.5 (

Table ***8***); therefore, ELM can be used. Whereas, For ELM, analysis is performed using final member sizes, with nominal (unreduced) stiffness. Notional loads are already applied to all gravity only combinations (the same as required for ELM). Shifting to ELM, moment frame K-factors have to be calculated.

Table 4: Notional loads

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Combo1	1.4D + 1.4Nx	*Notional lateral loads combined with gravity loads*
Combo2	1.2D + 1.6L + 0.5Lr + 1.2NDeadx + 1.6NLivex + 0.5NLiveRx	
Combo3	1.4D + 1.4Ny	
Combo4	1.2D + 1.6L + 0.5Lr + 1.2NDeady + 1.6NLivey + 0.5NLiveRy	
Combo5	1.4*D* – 1.4*Nx*	
Combo6	1.2D + 1.6L + 0.5Lr – 1.2NDeadx – 1.6NLivex – 0.5NLiveRx	
Combo7	1.4D – 1.4Ny	
Combo8	1.2D + 1.6L + 0.5Lr – 1.2NDeady – 1.6NLivey – 0.5NLiveRy	
Combo9	1.2D + 1.6Wx + 0.5L + 0.5Lr	
Combo10	1.2D – 1.6Wx + 0.5L + 0.5Lr	
Combo11	1.2D + 1.6Wy + 0.5L + 0.5Lr	
Combo12	1.2D – 1.6Wy + 0.5L + 0.5Lr	
Combo13	1.2D + 1.0Ex + 0.5L	
Combo14	1.2D – 1.0Ex + 0.5L	
Combo15	1.2D + 1.0Ey + 0.5L	
Combo16	1.2D – 1.0Ey + 0.5L	
Combo17	0.9D + 1.6Wx	
Combo18	0.9D – 1.6Wx	
Combo19	0.9D + 1.6Wy	
Combo20	0.9D – 1.6Wy	
Combo21	0.9D + 1.0Ex	
Combo22	0.9D – 1.0Ex	
Combo23	0.9D + 1.0Ey	
Combo24	0.9D – 1.0Ey	

Table 5: Drift for Serviceability Limit State Strength Controlled Braced Frame Design

Level	Deflection 10-yr wind, δ(in)	Story Drift 10-yr wind, δ (in)	Drift Index
Roof	0.825	0.079	H/1901
10	0.746	0.088	H/1709
9	0.656	0.089	H/1685
8	0.569	0.091	H/1650
7	0.478	0.089	H/1656
6	0.388	0.085	H/1690
5	0.299	0.080	H/1764
4	0.214	0.073	H/1877
3	0.134	0.073	H/2058
2	0.061	0.061	H/2451

Table 6: Drift for Serviceability Limit State Strength Controlled Moment Frame Design

Level	Deflection 10-yr wind, δ(in)	Story Drift 10-yr wind, δ (in)	Drift Index
Roof	3.43	0.13	H/1174
10	3.31	0.21	H/709
9	3.09	0.27	H/551
8	2.82	0.31	H/483
7	2.51	0.35	H/435
6	2.17	0.37	H/403
5	1.79	0.38	H/390
4	1.14	0.40	H/377
3	1.01	0.41	H/366
2	0.60	0.06	H/249

Table 7: Drift for Serviceability Limit State Strength Controlled Moment Frame Design

Level	Deflection 10-yr wind, δ(in)	Story Drift 10-yr wind, δ (in)	Drift Index
Roof	3.12	0.127	H/1178
10	2.99	0.211	H/710
9	2.78	0.272	H/552
8	2.51	0.310	H/484
7	2.20	0.344	H/436
6	1.86	0.371	H/404
5	1.49	0.375	H/400
4	1.11	0.385	H/400
3	0.737	0.362	H/414
2	0.374	0.374	H/401

Table 8: Second-Order to First-Order Drift Ratio

Level	**Δ2nd/Δ1st**
Roof	1.23
10	1.29
9	1.34

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8	1.38
7	1.42
6	1.45
5	1.47
4	1.47
3	1.47
2	1.49

VII. CONCLUSION

It is nearly impossible for a structure member designed using any method to have a capacity equivalent to the assumed loading conditions the structure system may encounter in its service life. Such consideration has been developed in AISC, which resulted in safety factors, load factors, and resistance factors. A method that does not account for all types of loads, including their magnitudes, in addition to ignoring geometric imperfections and construction flow, the same method of analysis provides conservative results. With these facts, a method that offers an unsafe design with failure characteristics, strict boundaries must be specified for its implementation.

In the case study, this paper details the simplicity of DAM by demonstrating with the design of a ten multistory building in Saap software. Furthermore, the following notes are highlighted:

- All loads and conditions that work to destabilize the structure system should be included in the structural model. Forgetting to add effects of all gravity columns that rely on that frame for support destabilize the frame.
- In the DAM, the story out-of-plumbness (Δ_o) needs to be accounted for an analysis model.
- Notional loads are likely to be applied to the direction that requires the most destabilizing effects.
- Reduced stiffness is only used in strength analysis. Whereas, unreduced stiffness is used for serviceability checks.

The result from the design analysis check revealed that the smooth and quicker procedures of the DAM application, including accounting for second-order analysis, prove whether the ELM is feasible.

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