CRITICAL EVALUATION OF METAL BUILDING SYSTEMS SUBJECTED TO EXTREME WIND LOADS

by

AJAY SHANKER, B.E., M.Sc.

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CHAPTER I
INTRODUCTION

1.1 Problem Statement

A substantial number of metal buildings are damaged or destroyed in extreme winds each year. Hurricane Hugo, which struck Charleston, South Carolina, in September 1989, damaged a number of warehouses built with steel frames and metal skin (McDonald and Smith 1990). The damage to metal buildings appears to be due to lack of coordination between designer and dealer/contractor, and poor construction practices (Perry et al. 1989).

Researchers involved in wind damage investigation have observed that some metal buildings perform adequately, while others are totally destroyed. This fact suggests the problem is not necessarily an underspecification of wind loads, but a problem with construction practice and marginal structural systems (Perry et al. 1989, Surry 1989, Meecham et al. 1987). Typically, wind first removes sheet metal panels, then purlins and girts become unsupported and buckle, leaving the frames unsupported; the frames then become laterally unstable and may fail at much less than the design load (Ellifritt 1981). Other omissions and structural deficiencies that may start the process of progressive collapse are listed below. The following reasons have been documented for the failure of metal buildings (Perry et al. 1989, Sparks et al. 1988, Mehta et al. 1983, Minor et al. 1977).

1. Stripping of sheet metal from walls and roof.
2. Omission/inaction of flange bracing.
3. Damage at steel/masonry facade.
4. Failure of anchoring system.
5. Failure of doors, glazing and perforation by wind borne missiles and debris.
6. Collapse of structure because of connection and column failure.
7. Strut purlin failures in the end-bay.

Although the reasons cited above for unsatisfactory performance of metal building systems have been known for some time, few studies have attempted to quantify these failure modes or construction deficiencies (Mehta 1984, Minor 1984, Sparks 1987). The designer has no information regarding these construction deficiencies and failure modes. The need for such research, which enables the designer to design structural systems having greater capacity in the load path in the event of overloading has been emphasized by many researchers (Perry et al. 1989, Bihr 1989, Mehta 1984, Mehta et al. 1983, Minor 1984, 1983, 1978, Minor et al. 1979, 1972, Walker et al. 1975). Further, the need for modeling human errors in structural design and construction has been demonstrated by a working group of ASCE (ASCE 1986).

Post-storm investigations have provided some knowledge of metal building system performance (Mehta 1984, Mehta et al. 1983, Minor et al. 1977), but knowledge has been slow finding its way into the MBMA standard. The MBMA standard recommends wind loads based on the assumptions that all structural components of the building will be installed as specified, and the building will be erected as per the design. However, in practice many omissions by contractors have been observed. These aspects have not been addressed in the MBMA standard. The standard clearly states that the translation of the knowledge gained from post-disaster investigations into codes of practice continues to be a formidable task (MBMA 1986).

During the past two decades, research activity in wind engineering and wind damage mitigation had been concentrated on finding the appropriate values for wind-induced loads. Various wind tunnel and full-scale studies (Davenport et al. 1977, 1978, Cermak 1977, Best and Holmes 1978, Mehta et al. 1988) have concentrated on the load side of the equation. Little has been done on the resistance (response) side of the equation (Perry 1987). A major effort to look at the response of the elements and assemblages subjected to wind loads is needed to mitigate the wind damage (Perry 1987).

In attempting to gain a competitive edge with other building systems, (e.g., timber and precast concrete) the metal building industry employs optimization techniques for the design of main frames, leaving very little margin of safety. No one has looked at the consequences of omissions during the construction process and the failure modes observed in post-storm investigations.

Strategies for mitigating the wind damage suggest research needs in several areas (Perry et al. 1989). The research reported herein attempts to address three aspects of the problem:

1. Problems of the designer/manufacturer.
2. Problems of the dealer/contractor.
3. Problems relating to inspection and construction quality.

1.2 Objectives

The general objective of this study is to explore ways to solve the problems mentioned above. The following problems associated with extreme winds are addressed in this study.

1. Breach of the building envelope.
2. Omitted bracing of compression flanges.
3. Purlin anchorage failures.
4. Unexpected overload due to high winds.
5. Effect of tapered members.

To study the effects of above mentioned problems a low-slope frame having a roof slope of .5/12 and a high-slope frame having a roof slope of 7/12 are considered in this study. Frames are designed for enclosed and partially enclosed conditions for both roof slope cases according to MBMA 1986. The frames are designed for the interior zone using 100 mph design wind speed. A 120'-span and 30'-bay spacing is used. The frames are adequate for all applicable wind loading cases. Columns and rafters are tapered to achieve maximum economy.

The main objective of this study is to evaluate the resistance capacity of the steel gable frames in light of construction deficiencies and failure modes observed in post-storm investigations.

The following specific objectives are considered:

1. A breach in the building envelope is studied by subjecting the enclosed designs to partially enclosed wind loadings. Wind loads for all the loading cases for the partially enclosed condition are applied to the Low-slope and High-slope frames. Maximum increases in knee moment, ridge moment, vertical and horizontal reactions at column base are

computed. Increases in the bending moments along the column and rafter are studied. Locations on the column and rafter where AISC unity checks are exceeded are identified. Weights are also computed for frames designed for enclosed and partially enclosed conditions. Economic aspects of the designs for enclosed and partially enclosed conditions are examined by using industry-specified cost factors for the material and fabrication of the main frames.

2. For the frames designed for partially enclosed wind loads, it is assumed that two adjacent flange braces are omitted by the construction personnel. The braces were omitted in the region of maximum negative moment. The laterally unsupported section is replaced by an equivalent braced section using AISC criteria. The frames are then subjected to same partially enclosed wind loads. Changes in the moments and AISC unity check along the column and rafter are studied.

3. For the frames designed for partially enclosed wind loads, it is assumed that, on the left rafter, purlin anchorage failure occurs for four consecutive purlins. The purlins, therefore, become ineffective in transferring their share of wind uplift force to the rafter. Redistribution of uplift is assumed as permissible and causing the most undesirable effect. Changes in the moments and the AISC unity check are studied along the length of columns and rafters.

4. Wind loads are computed for 110, 120, 130 and 140 mph winds for all loading cases for the partially enclosed condition. Wind loads due to increased wind speeds are then applied to frames designed for 100 mph wind. The changes in the moments, vertical and horizontal reactions at the base, and the AISC unity check are studied for each increment in the

wind speed along the column and the rafter. Regions vulnerable for reaching higher stresses are identified. Frames are also designed, for both the roof slopes, to resist 110, 120, 130, and 140 mph winds. Weights of the frames for higher wind speeds are computed. Economic aspects of designing for higher wind loads are examined by using an industry-specified cost factor for the material and fabrication for the main frames.

5. Main framing pressure coefficients in MBMA 1986 have been derived using two-hinge and three-hinge frames having prismatic members. Whether coefficients based on prismatic members provide adequate or safe loads for tapered frames is examined. Two-hinge and three-hinge uniform frames are subjected to partially enclosed wind loads. Design parameters (e.g., knee moment, ridge moment, vertical and horizontal reactions at the base) are compared with the values obtained for tapered frames. The adequacy of the use of coefficients based on the assumption of prismatic members is examined.

1.3 Expected Results

For the manufacturer/designer results of this study will highlight the importance of the failure modes observed in post-storm investigation. Quantitative analysis indicating the increases in the moments and stresses for the failure modes, e.g., breach of envelope, purlin anchorage failures, omission of flange bracing, or overload, will alert the manufacturer/ designer of the possible ranges of the overstresses the building may experience in extreme wind storms. Manufacturer/designer will then be able to design building systems capable of resisting these overstresses.

For the dealer/contractor, the comparison of enclosed and partially enclosed cases will highlight the importance of the selection of good quality doors, windows and glazing. The analysis, indicating the ranges of overstress due to purlin anchorage failures, and omission of flange bracing will stress the level of care to be exercised in installing these components. Further, the results of the study will stress the need for the cooperation and close coordination of all parties involved in the construction.

Most metal buildings are designed to satisfy requirements of the MBMA standard. Post-storm investigations suggest that some metal buildings designed to satisfy the minimum requirements of MBMA do not perform well in wind storms. The study highlights the potential economic benefits of adopting more conservative design criteria than specified by MBMA.

1.4 Selection of Design Parameters

This study is done on the typical designs of single-span tapered gable frames. The design parameters and study tasks are carefully chosen so that the results may be useful to designers, contractors, and owners involved in the metal building industry. The limitations, and the reasons for choosing certain design parameters and study tasks, are discussed subsequently.

1. This study is limited to frames designed for .5/12 and 7/12 roof slope buildings. Buildings are categorized by MBMA according to roof slope, e.g., $0\text{-}10^0$, $10^0\text{-}30^0$, and $30^0\text{-}45^0$. The pressure coefficients remain the same for roofs falling within a particular roof slope category. Roof

slopes considered in this study cover 0-10^0 and 30^0-45^0 roof slope cases. Fifty to sixty percent of single-span metal building construction falls in the category of 0-10^0 roof slope (Perry 1989).

2. MBMA 1986 categorizes metal buildings in three types, i.e., enclosed, partially enclosed and open. In this study the effect of breach of enclosed buildings is studied. The effects of omitted compression flange bracing, purlin anchorage failures, overload, and use of tapered members are studied for partially enclosed buildings. The reasons for choosing the partially enclosed case for studying these conditions are now discussed. Common industry practice is to design metal buildings as enclosed buildings. However, recent studies (Perry 1989) recommend that proper interpretation of the industry standard requires that a high percentage of metal buildings be designed as partially enclosed. Therefore, it was considered appropriate to study the effects of these conditions on partially enclosed buildings.

3. There are only seven or eight major reasons for wind damage to these buildings as per disaster investigation studies. Effects of four of these reasons on the design of main frames are examined in this study.

4. Study is done on four typical designs for low and high roof slopes for the enclosed and partially enclosed buildings. Results of this study can be considered as typical for frames having the same design parameters, e.g., span, roof slope etc. For the design loads as per MBMA 1986 and design stresses as per AISC, most of the design programs will lead to the same web and flange plate sizes, and taper ratios for achieving minimum weight of the main frame. Therefore, major changes in the design of

frames by different manufacturers, using different design programs, are not expected.

5. This study is limited to single-span metal buildings that are comprised of tapered gable frames, c or z cold-formed purlins and girts, and sheet metal. However, it may be noted that thousands of metal buildings that are comprised of above noted structural components, worth billions of dollars are erected every year.

6. A design wind speed of 100 mph is considered in this study. A review of the hurricane prone regions listed in Appendix A reveals design wind speed in these areas lies between 80-100 mph.

A limited number of further studies of this type can generate enough information to help the designer to design better wind-resistant metal buildings than at present. Further, the information obtained from such studies can be used to affect suitable changes in the industry standard.

Metal buildings sustain significant damage in windstorms. The purpose of this study is to find ways to mitigate this wind damage. The importance of mitigating wind damage is because the metal building industry has a large marketshare of the industry and continues to grow. Mitigation strategies must be selected to the extent possible that will not diminish the market share or slow down growth of the industry. Marketshare and sales statistics of metal building systems are discussed subsequently.

1.5 Marketshare and Sales of Metal Building Systems

Pre-fabricated (or "pre-engineered") metal building systems provide some of the lowest costs and fastest construction times available today. Over 50% of non-residential low-rise buildings (below 150,000 square feet) erected in the United States today are pre-engineered metal structures (MBMA 1989, Ubios 1986). During the last 40 years there has been an exponential increase in the use of metal building systems. Combined 1989 sales for MBMA's member manufacturers have touched an all time high of $1.68 billion (MBMA 1989). A metal building system typically represents about 20% of the total project cost. Thus, 1989 MBMA sales accounted for some $8.4 billion of new in-place construction (MBMA 1989). For the one- and two-story community, commercial and industrial buildings up to 150,000 square feet, the market share for metal building systems stood at 57.1% in 1989. About 50% to 60% of metal building construction falls into the category of single-span rigid gable frames made of tapered members (Perry 1989).

Thus, it can be estimated that, at present, about $800 to $900 million worth of new metal building systems comprised of single-story gable frames with tapered members leading to about $4 billion worth of total in-place construction is added every year to the commercial and industrial markets. This research work is related to single-span rigid gable frames with tapered members. All subsequent information relates directly to such form of framing system and will after this be referred to as the metal building system.

1.6 Structural Components of Metal Building Systems

Figure 1.1 shows the typical framing system and various structural components of a metal building system. The main frames are usually welded up from plate and bar stock having a yield strength of 50 ksi. Standard hot-rolled sections are generally not used. The frames are fabricated with pre-punched splice plates for easy field bolting. Typically, all connections between members in a rigid frame are moment connections made with end plates. Roofs are typically fabricated from light gage steel, corrugated for extra strength. Roof sections are supported by purlins (secondary structural members) that transfer the load from the roof panels to rigid frames. Purlins play a dual role: anchoring the panels and supporting the roof load. Sheet metal for roofing is generally attached to purlins by self-tapping screws. Purlins are usually cold-formed z-sections.

1.7 Summary of the Remaining Chapters

The general procedures for the design of the frames according to MBMA criteria and procedures for evaluating the effects of failure modes are explained in Chapter II. Details of design loads and the detailed designs of two frames with the detailed design procedures are given in Chapter III. Analysis and evaluation of effects of extreme winds are given in Chapter IV. Summary, conclusions and recommendations are given in Chapter V.

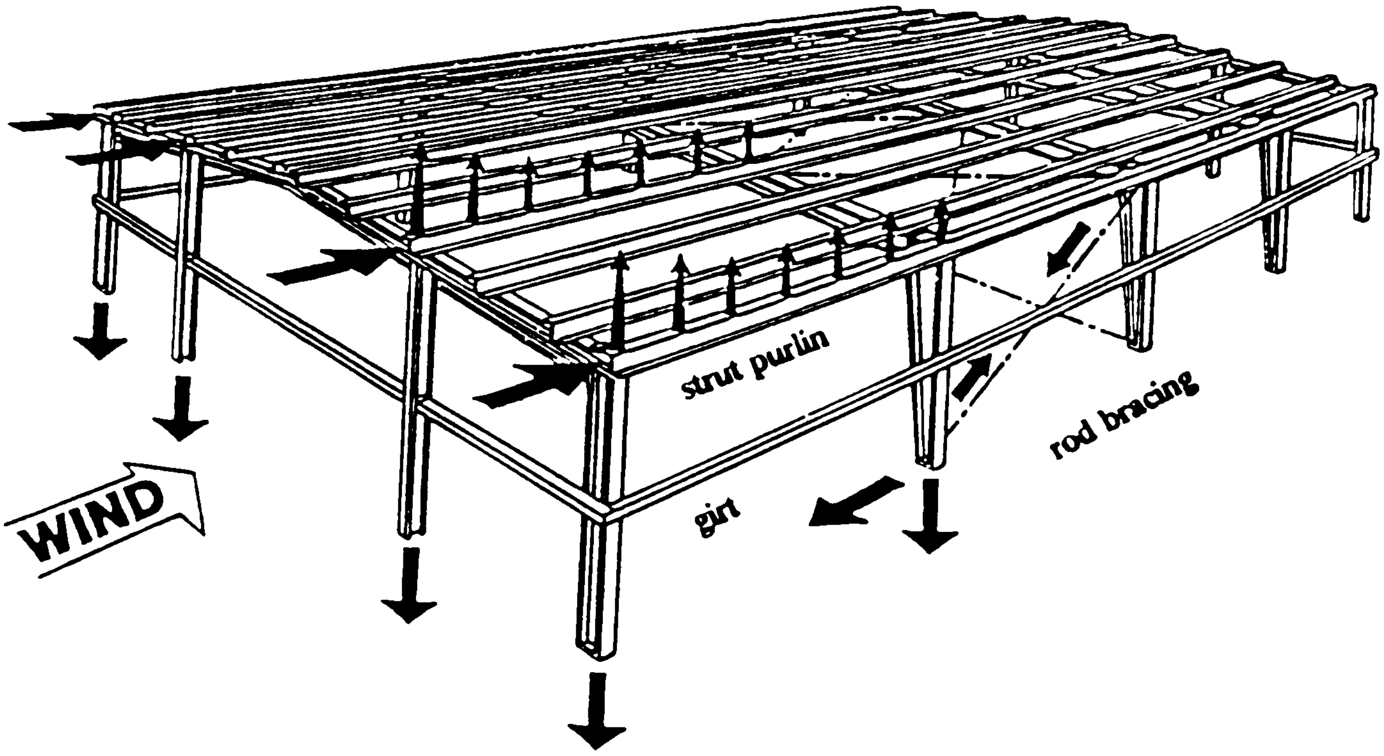


Figure 1.1: Typical Single-span Rigid Tapered Gable Frame Buildings.

CHAPTER II
PROCEDURES

2.1 General

This chapter describes the procedures followed in conducting this study. The procedures followed in each of the five study tasks are briefly described in this chapter. Preliminary studies showed that the roof slope is the most important parameter affecting the wind flow pattern and the wind pressures on the building envelope. Two roof slopes are considered that bound the range of slopes used in building practice. Frames are designed for the roof angle of 2.4^0 (.5/12 roof slope, 0.5 vertical to 12 horizontal) and roof angle of 30.25^0 (7/12 roof slope, 7 vertical to 12 horizontal). Frames are designed and are optimized for weight, using DESIGN program (Synercom 1988). This program is presently being used in the metal building industry. The program employs the direct stiffness method and allowable stress design for structural analysis and design. Frames are designed according to MBMA 1986 and AISC 1980 design criteria.

2.2 Breach in Building Envelope

Breach of building envelope is probably the most prevalent cause of damage to metal buildings by extreme winds. A breach can occur due to stripping of sheet metal panels, failure of doors and windows, or by the perforation of walls or roof by windborne missiles. A breach in the building envelope changes the magnitude and distribution of internal wind pressures and affects the wind loads that must be carried by the structural components.

Design procedures in most codes and standards treat buildings as enclosed, partially enclosed, or open. In short, an enclosed condition means that the building encloses a space and has a uniform distribution of openings in the building envelope. The windows, doors, and other building accessories designed to resist wind pressures need not be considered as openings. A partially enclosed condition means that a particular wall has more openings than total openings in the rest of the walls. An open condition means that the walls of the building are 80% open. A detailed description of these conditions is given in chapter III.

Present industry practice is to design most of the metal buildings as enclosed buildings. To evaluate the effects of breach in the building envelope, main frames are designed for the enclosed condition. Frames are then subjected to the wind loads for the partially enclosed conditions. MBMA 1986 prescribes two or three sets of the pressure coefficients leading to four or six loading cases for the partially enclosed conditions. The standard specifies that the design be provided for the most critical condition. Therefore, wind loads for all loading cases for the partially enclosed condition are considered. The maximum changes in the design moment profile (max of DL+LL and DL+WL) and the unity check are computed along the lengths of the rafter and column. The unity check means that the combined effect of computed axial and bending stresses is less than or equal to the allowable stresses. Changes in the vertical and horizontal reactions on a base connection are also computed.

2.3 Omitted Bracing of Compression Flanges

Many cases of failure have been reported in which the bracing for the compression flange of rigid frames is omitted or is improperly installed by the contractor (Perry 1989). This situation arises as the flange bracing is installed after the all erection processes, e.g., bolting of main frames, installation of purlins and girts are complete. Flange bracing interferes in the bolting process, therefore it is installed at the end by the erectors. Because contractors and erectors may not appreciate the importance of the bracing, many times it is left out. The bracing is more important in resisting wind loads rather than gravity loads. Hence its need at the time of erection may not be appreciated by the constructors.

To evaluate the effects of the omitted bracing of the compression flanges, it is assumed in this study that the frame loses the lateral restraint of the inner compression flange at two adjacent points in the region of maximum negative moment. The allowable bending stress of the unbraced segment is computed. An equivalent section that is having bracing at the original points and having the same bending resistance as that of the unbraced segment is then designed. To match the bending resistance of the unbraced segment and the equivalent braced segment a smaller compression flange for the equivalent section is used. The equivalent section is arrived at by trying different smaller compression flanges. The AISC procedure and the design charts for the tapered members (Lee et al. 1981) are used to compute the stresses and for designing these segments. The equivalent section is replaced in the original frame and the modified frame is then subjected to wind loads as used in the original design. Changes in the moment profile and the unity check are then computed along the column and rafter.

2.4 Purlin Anchorage Failures

Purlins are usually bolted to rafters using A307 low-carbon steel bolts. Purlin anchorage failures are common in hurricanes and tornadoes. Purlin anchorage failures occur due to mispunched holes, tearing of the sheet metal near the bolt holes, and due to fatigue effects. When purlin anchorage failure occurs, the wind load pattern is altered from that assumed in the original design of the rafter.

To evaluate the effects of purlin anchorage failures, it is assumed in this study that anchorage failure of four consecutive purlins occur on a rafter in a wind storm. The loads of three purlins are assigned to one adjacent purlin on one side and the load of the fourth purlin is assigned to the adjacent purlin on the other side. If there is no breach of the roof surface the wind continues to exert the same total uplift, the loads for purlins whether inactive or intact conditions are kept the same as per MBMA 1986. The frames are then analyzed using this redistributed load. The changes in the moment profile and the unity check along the column and rafter are plotted.

2.5 Unexpected Overload due to High Winds

When an owner elects to construct a metal building to minimum standards, he should be aware that the building could be overloaded by high winds one or more times during the life of the building. He should be aware of the consequences of the overloading and weigh the possibility of damage to his facility against the additional costs of higher design loads.

Since at high wind speeds the building may lose wall or roof panels, frames designed for the partially enclosed condition are used to evaluate the effects of overload. Frames are designed for low-slope as well as high-slope for a partially enclosed building and 100 mph wind load. These frames are then subjected to wind loads on a partially enclosed building for 110, 120, 130 and 140 mph winds. Changes in moment and the unity check at many points along column and rafter are computed. Failure of anchorages at column base are examined by computing changes in the vertical and horizontal reactions at column base. The low-slope and high-slope frames are also designed to resist wind speeds of 110, 120, 130 and 140 mph. The weights for the frames suitable for resisting the increasing wind speeds are computed.

2.6 Tapered Versus Prismatic Members

Researchers at the University of Western Ontario conducted extensive wind tunnel studies for MBMA to obtain the wind pressure coefficients for the design of metal buildings. UWO used an approach that obtains envelope values for all possible wind directions relative to the building. The procedure requires the use of influence coefficients for the building rigid frames. In the UWO study prismatic rafters and columns are assumed. In practice the rafters and columns are normally tapered to reduce the steel requirements. This phase of the study examines the effects of considering tapered members instead of prismatic members.

To evaluate the effects of the use of tapered members a comparison of design parameters for prismatic and tapered frames is made. Only relative moments of inertias for the prismatic and tapered frames were used to make these comparisons. The design parameters, e.g., moments at knee and ridge, vertical and horizontal reactions at a base connection are computed for prismatic as well as tapered frames. The two-hinge and three-hinge prismatic frames are considered as in the UWO study. Design parameters are then computed for two-hinge and three-hinge prismatic frames. Comparison is made by computing the same design parameters for the two-hinge tapered frames used in this study. By comparing the ranges of the design parameters for the prismatic and tapered frames suitability of the assumption of prismatic members is evaluated.

Designs of the low-slope and high-slope frames are described in the next Chapter. Details of the studies are described in Chapter IV.

CHAPTER III
DESIGN OF RIGID FRAMES

3.1 Introduction

This Chapter describes the design of the two typical metal building frames used in this study. The frames are designed using standard MBMA design practice. An industry produced computer program (courtesy of Synercom Technology) is used to perform the analysis and member selection. Use of the program assures that the frames studied are typical of those produced in industry practice. A validation of the computer code is performed prior to using it for the design of the frames. A discussion of the validation procedures and conclusions is presented in Appendix B.

Prior to detailed description of the two frame designs, the various aspects of the MBMA design procedures are described and discussed in this Chapter. An understanding of the approach and rationale for the design of metal building rigid frames is presented as background for the analyses described in Chapter IV.

3.2 Dead and Live Loads as per MBMA 1986

Dead loads for the design of main framing for metal buildings are comprised of self-weight of the framing, weight of roof covering and secondary framing, i.e., purlins, insulation, and any other loads incorporated into the system to be permanently supported by main framing. The four most common forms of insulation for metal buildings are: flexible blanket, rigid board insulation, spray-on insulation, and foamed-in-place usually urethane or polysterene foam weighing from one psf to four psf (MBMA 1981, Ubois 1988). Dead loads for sheet metal, purlins, and insulation commonly used for the design of metal building systems are tabulated in Table 3.1.

Roof live loads are the loads that are produced (1) during maintenance by workers, equipment, and materials, and (2) during the life of the structure by movable objects, but not including wind, snow, seismic or dead loads. Roof live loads for the design of metal building systems are tabulated in Table 3.2. To account for the fluctuating nature of and averaging effects for roof live loads, design roof live loads decrease as the tributary area increases (Table 3.2).

3.3 Wind Loads as per MBMA 1986

Wind loads for the design of main framing are computed by the action of design wind pressure over the tributary area of the main framing. The design wind pressure is calculated using the following formula:

$$p=q(GC_p)$$

where

p= Design wind pressure in pounds per square foot.

q= Velocity pressure in pounds per square foot (psf) as set forth in Table 3.3. Basic wind speed is according to ANSI A58.1-1982 or subsequent revision.

Table 3.1

Dead Loads of Standard Roofing Materials for Metal Buildings

Dead Loads of Standard Roofing Materials for Metal Buildings			
Sheet Metal			
Gage Thickness	Equivalent Thickness (inches)	Wt in psf (standard sheet)	Wt in psf (Galvanized sheet)
18	0.0516	2.00	2.15
20	0.0396	1.50	1.65
22	0.0336	1.25	1.40
24	0.0276	1.00	1.15
26	0.0217	0.75	0.90
Cold formed Purlins and Girts			
Type of Section	Size	Weight	
C Section	8" x 3" x 14 Ga	3.89 lbs/foot	
Z Section	8" x 3" x 14 Ga	3.89 lbs/foot	
Insulation			
Polystyrene Foam	0.2 lbs/sq foot per inch of thickness		
Urethane Foam with skin	0.5 lbs/sq foot per inch of thickness		
Fibre Board	1.5 lbs/sq foot per inch of thickness		
Fibre glass	1.1 lbs/sq foot per inch of thickness		
Cellular glass	0.7 lbs/sq foot per inch of thickness		

Table 3.2

Design Roof Live Loads* for Metal Building Systems

Roof Slope	Tributary Loaded Area in Square Feet for any Structural Member.		
	0 to 200	201 to 600	Over 600
Flat or rise less than 4:12	20	16	12
Rise 4:12 to less than 12:12	16	14	12
Rise 12:12 and greater	12	12	12

*In pounds per square foot (psf) of horizontal roof projection

Table 3.3

Velocity Pressure, q, in Pounds per Square Foot (psf)

		Fastest-mile Wind Speed in mph (V)				
		70	80	90	100	110
	0					
Mean	15	10.0	13.1	16.6	20.4	24.7
Roof	16	10.2	13.3	16.9	20.8	25.2
Height	17	10.4	13.6	17.2	21.2	25.6
'H'	18	10.5	13.8	17.4	21.5	26.1
in	19	10.7	14.0	17.7	21.9	26.5
feet	20	10.9	14.2	18.0	22.2	26.8
	22	11.2	14.6	18.5	22.8	27.6
	24	11.5	15.0	18.9	23.4	28.3
	26	11.7	15.3	19.4	23.9	28.9
	28	12.0	15.6	19.8	24.4	29.6
	30	12.2	15.9	20.2	24.9	30.1

where (1) $q = 0.00256\, V^2 \left(\frac{H}{33}\right)^{\frac{2}{7}}$.

A single value of q is used for the entire building.

(2) V = Fastest-mile wind speed in miles per hour determined from ANSI A58.1-1982.

(3) H = Mean height of roof above ground or 15 feet whichever is greater. Eave height may be substituted for mean roof height if roof slope "a" is not more than 10°.

GC_p= Peak combined pressure coefficient for the main framing as given in Table 3.4. The coefficients given in this table represent the peak combined external and internal pressure coefficients and include the gust response factor.

An edge strip, Z, is a strip all along the periphery of the building. The width of the edge strip is defined as 10% of the minimum width or 0.4H, whichever is smaller, but not less than 0.04 B or 3 feet. End Zones extend inwards from the end walls. The width of an end zone is the greater of 20 feet or 2Z. All areas not within the end zone are considered in interior zone. End Zones are indicated in Figure 3.1.

According to MBMA the following definitions apply for computing the wind loads.

Openings- -Those areas in the building envelope (wall, roof surfaces) which do not have a permanently attached means for effective closure.

Enclosed Buildings- -Structures that enclose a space and have a uniform distribution of openings in the building envelope (wall, roof surfaces). Windows, doors, and other building accessories designed to resist wind pressures need not be considered as openings.

Partially Enclosed Buildings- -An enclosed building in which the ratio of total openings, A_o/A_g, in the dominant wall (defined as the wall containing the largest ratio of openings) satisfies the conditions:

$$\frac{A_o}{A_g} > 0.05 \quad \text{and} \quad \frac{A_o}{\Sigma A_i} > 1.05$$

where

A_o = sum of openings in dominant wall in square feet (ft^2).

Table 3.4

Main Framing Coefficients GC_p for Transverse Direction

Roof(2) Angle "a"		Load (1) Case	End Zone Coefficients				Interior Zone Coefficients			
			1E	2E	3E	4E	1	2	3	4
Enclosed Building	0<a<10	I	+0.50	-1.40	-0.80	-0.70	+0.25	-1.00	-0.65	-0.55
		II	+0.90	-1.00	-0.40	-0.30	+0.65	-0.60	-0.25	-0.15
	10<a<30	I	+0.70	-1.40	-1.00	-0.95	+0.40	-1.00	-0.75	-0.70
		II	+1.10	-1.00	-0.60	-0.55	+0.80	-0.60	-0.35	-0.30
	30<a<45	I	-0.75	-1.40	-0.80	-0.75	-0.70	-1.00	-0.65	-0.70
		II	+0.60	+0.10	-0.80	-0.75	+0.45	+0.05	-0.70	-0.65
		III	+1.00	+0.50	-0.40	-0.35	+0.85	+0.45	-0.30	-0.25
	a=90	I	-0.70	-1.00	-0.65	-0.70	-0.75	-1.40	-0.80	-0.75
		II	+0.45	+0.45	-0.65	-0.65	+0.60	+0.60	-0.75	-0.75
		III	+0.85	+0.85	-0.25	-0.25	+1.00	+1.00	-0.35	-0.35
Partially Enclosed	0<a<10	I	+0.10	-1.80	-1.20	-1.10	-0.15	-1.40	-1.05	-0.95
		II	+1.00	-0.90	-0.30	-0.20	+0.75	-0.50	-0.15	-0.05
	10<a<30	I	+0.30	-1.80	-1.40	-1.35	0.00	-1.40	-1.05	-1.10
		II	+1.20	-0.90	-0.50	-0.45	+0.90	-0.50	-0.25	-0.20
	30<a<45	I	-1.15	-1.80	-1.20	-1.15	-1.10	-1.40	-1.05	-1.10
		II	+0.20	-0.30	-1.20	-1.15	-0.05	-0.35	-1.10	-1.05
		III	+1.10	+0.60	-0.30	-0.25	+0.95	+0.55	-0.20	-0.15
	a=90	I	-1.10	-1.40	-1.05	-1.10	-1.15	-1.80	-1.20	-1.15
		II	+0.05	+0.05	-1.05	-1.05	+0.20	+0.20	-1.15	-1.15
		III	+0.95	+0.95	-0.15	-0.15	+1.10	+1.10	-0.25	-0.25
Open	0<a<10	I		-0.70	-0.70			-0.70	-0.70	
		II		-0.30	-0.80			-0.30	-0.80	
	10<a<25	I		-0.70	-0.70			-0.70	-0.70	
		II	(3)	+0.70	-0.70	(3)		+0.70	-0.70	(3)
		III		+0.20	-0.90			+0.20	-0.90	
	25<a<45	I		-0.70	-0.70			-0.70	-0.70	
		II		+2.00	+0.30			+2.00	+0.30	

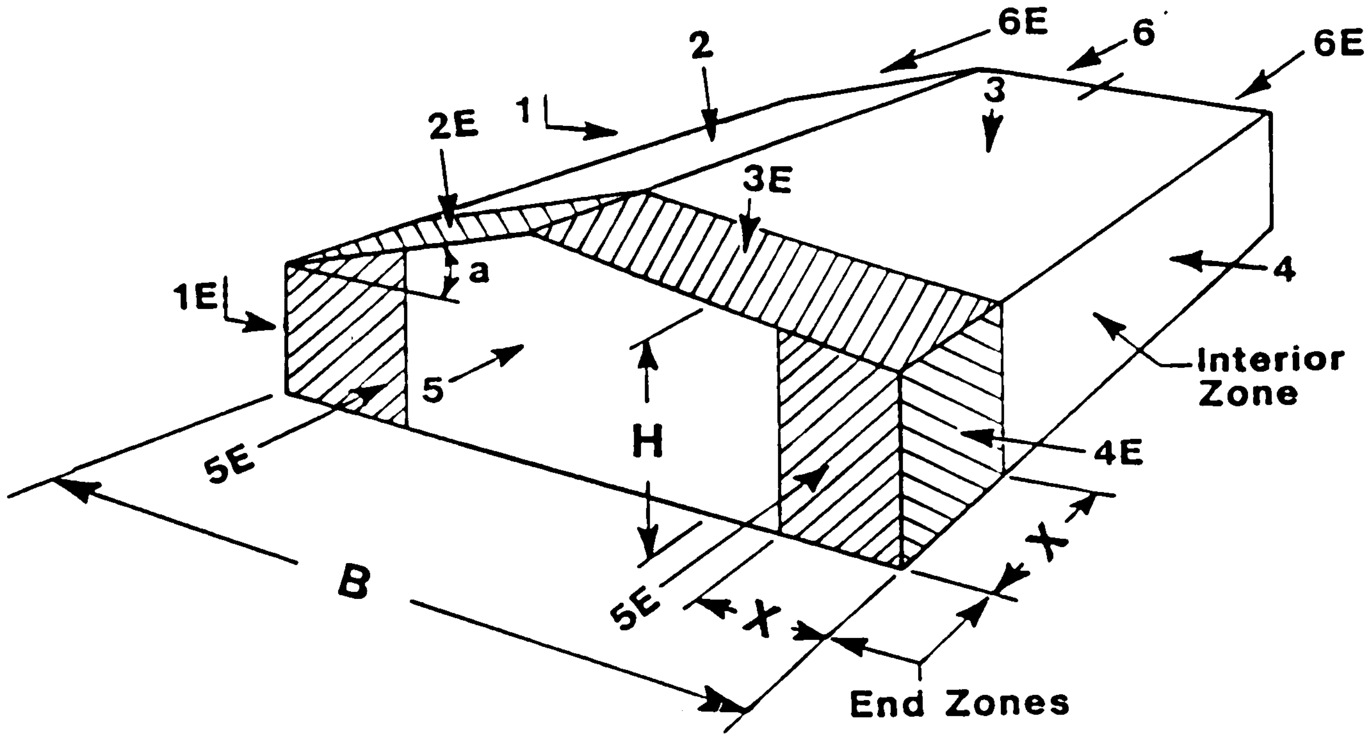


Figure 3.1 Zones of a Metal Building System for Main Frame Coefficients

$\sum A_i$ = sum of the openings in the remaining building envelope (wall, roof surfaces) in square feet.

A_g = gross area of dominant wall in square feet (ft^2).

Table 3.4 specifies the sets of coefficients, GC_p, for interior zone and end zone. Figure 3.2 and Figure 3.3 explain the development of these coefficients. These coefficients have to be multiplied by the velocity pressure to obtain the design wind pressure. In Figure 3.1 Zones 1 and 2 represent left and right longitudinal walls, and Zones 3 and 4 represent left and right roof Zones. Zones 1E through 4E are the corresponding end Zones. The coefficients are applicable for the main wind force resisting system in the transverse direction only. Where more than one load case exists for a given roof angle, framing shall be designed for the most critical condition.

The structural elements of a metal building system are designed to resist the loads contributed by their respective tributary areas (Lee et al. 1981, MBMA 1986). Therefore dead, live and wind loads will be computed for the tributary areas of each structural component.

3.4 Philosophy of Structural Analysis and Design

Tapered members are best suited to elastic allowable stress solutions based on elastic methods of analysis. While plastic design solutions could be carried out for structures composed of tapered members, the two ideas are philosophically in conflict (Lee et al. 1981). Tapered members are proportioned more or less so that they realize their allowable elastic stress at many cross-sections simultaneously. Plastic design on the other hand,

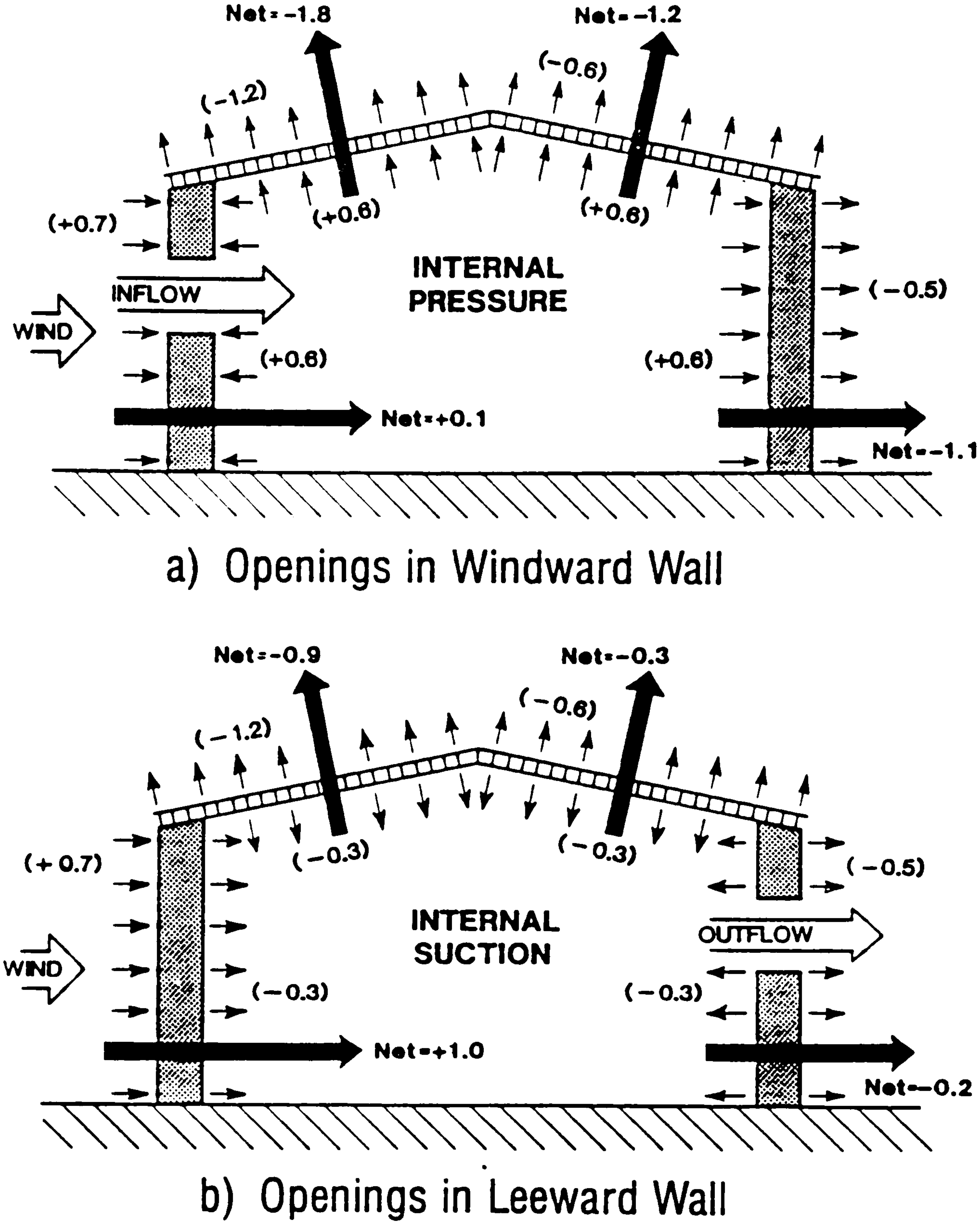


Figure 3.2 Derivation of Load Cases I and II (Table 3.4) for a Partially Enclosed Building with Dominant Openings in One Wall ($0^0<a<10^0$)

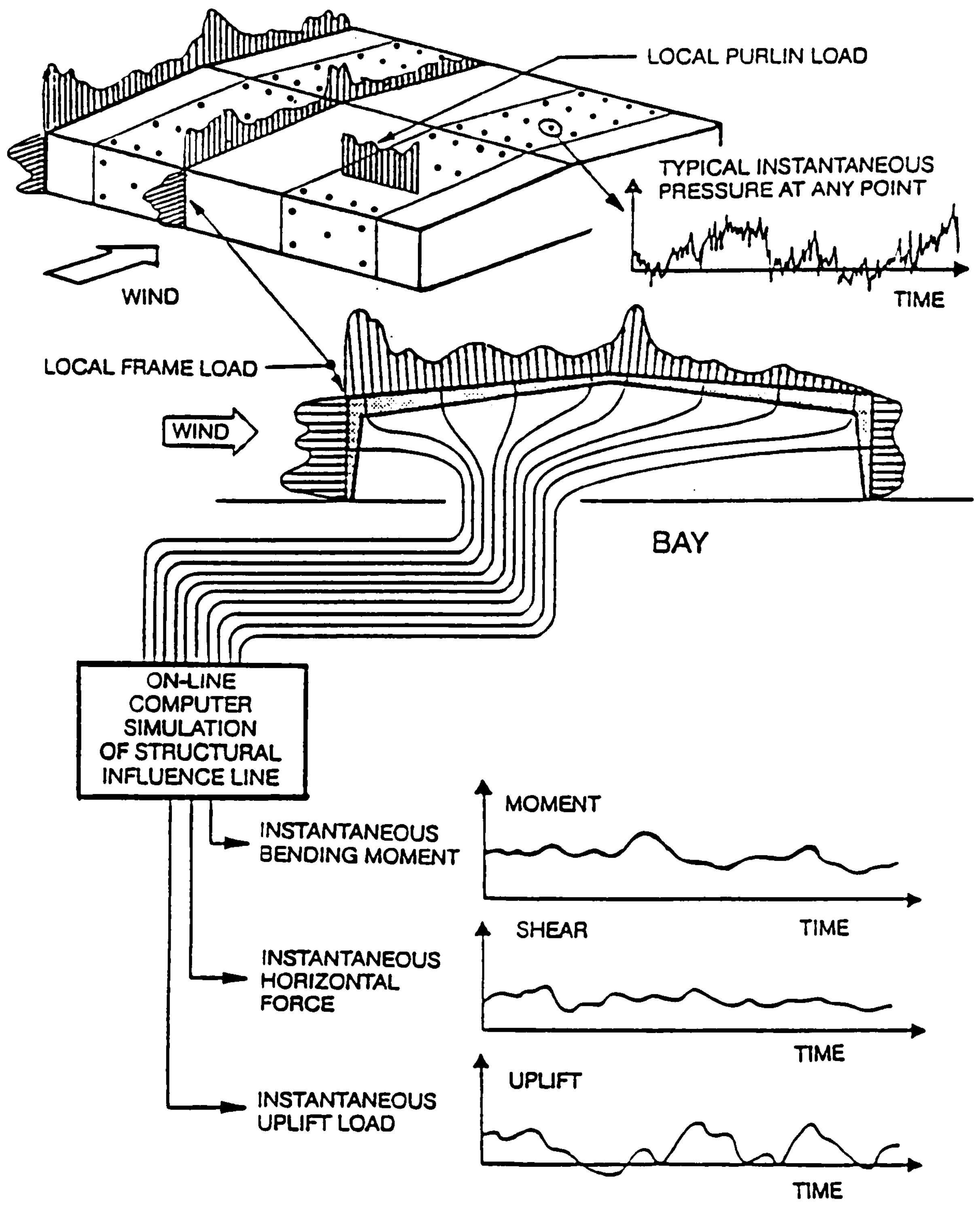
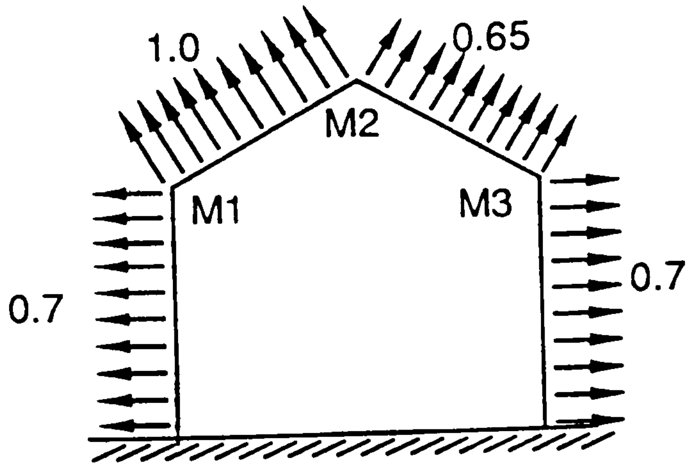


Figure 3.3 Development of Pressure Coefficients Using the Envelope Approach (After Davenport et al. 1979)

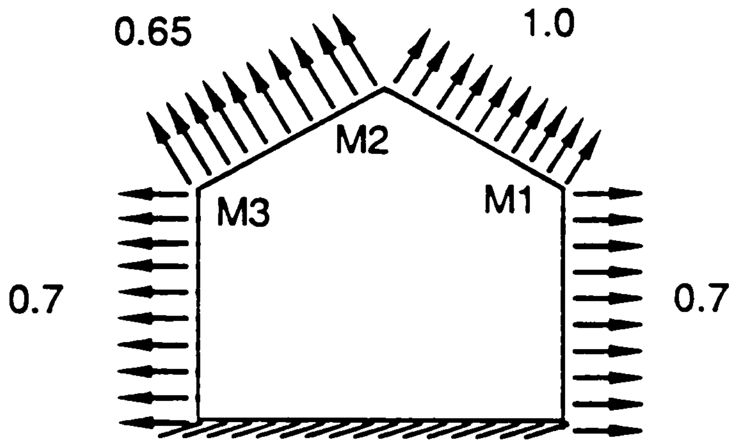
presumes early realization of inelastic action at few cross-sections, followed by inelastic rotation at those locations sufficient to allow redistribution of bending moments to other, less heavily stressed locations. Tapered sections preclude the early realization of inelastic action at the heavily stressed locations for a much larger range of load fluctuations as compared to prismatic members. Common industry practice is, therefore, to design tapered members, based on elastic methods of analysis (Lee et al. 1981). The DESIGN (Synercom 1988) program used in this research is based on elastic allowable stress design. This program implements the AISC design equations given in Appendix C. The analysis part of the program is based on the direct stiffness method for analyzing the plane frames. The program descritizes tapered members to be represented by the smaller prismatic segments having the sectional properties at the middle of the segment. Columns are broken down to about 1.9 feet length segments. Rafters are broken down to segments of about 3 feet. Experience has shown that adequate accuracy can be achieved by the use of smaller segments of such lengths (Lee et al. 1981). The accuracy of the program is validated by reproducing the moment diagrams for a tapered frame from the literature (Lee et al. 1981). A comparative statement of the results from the program and values from the literature is provided in Appendix B. The frames have been designed using commercially available plate stock of 50 ksi yield strength. These frames have also been optimized to reduce the weight as far as practicable. Common industry practice for the analysis and design of the frame is used.

A review of Table 3.4 indicates that pressure coefficients are different for the rafters in Zones 2 and 3. Coefficients are different for columns in Zones 1 and 4 also. For 7/12 roof slope there are three load cases leading to three designs for the left rafter and three designs for the right rafter. Common industry practice is to design only the left rafter and left column using the principle of symmetry, by switching the coefficients on rafters and columns. Maximum moments are then picked up at small intervals along the rafter for all six loading cases and the design is provided for the maximum moments at every point. The same design is then provided for the rafter on the right side. The use of symmetry is explained by an example.

Figures 3.4, 3.5 and 3.6 show pressure coefficients for the 7/12(30.25^0) roof slope frame for the enclosed case, interior zone (Table 3.4). Figure 3.4 shows the WLL and WLR loading conditions for the load Case I. Figure 3.5 shows WLL and WLR the loading conditions for the load Case II. Figure 3.6 shows the WLL and WLR loading conditions for the load Case III. It may be noted WLR coefficients are obtained by switching the coefficients of the WLL case as shown. As per MBMA 1986 criteria frames should be designed for all three load cases. Since both rafters should have the same dimensions, this technique of applying the loads from left and right can be used to pick up the worst moments for a rafter. Metal building industry also follows this technique of applying loads from left and right for picking up the worst moments for the design of a rafter (Synercom 1988). Moments M1 through M9 for the WLL cases, therefore, are also swapped, to obtain the moments for the WLR cases.

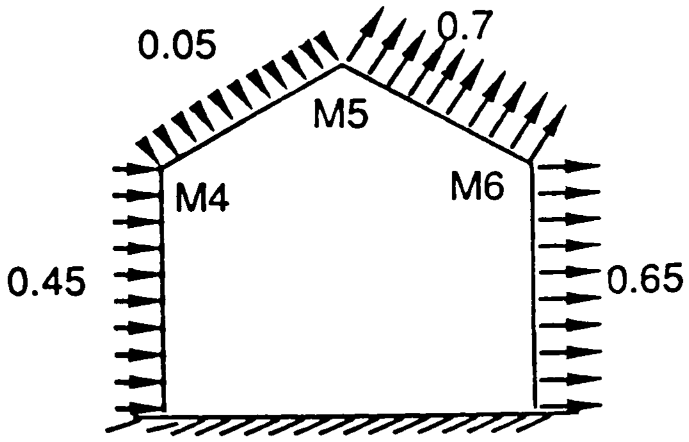


WL1 (Case I)

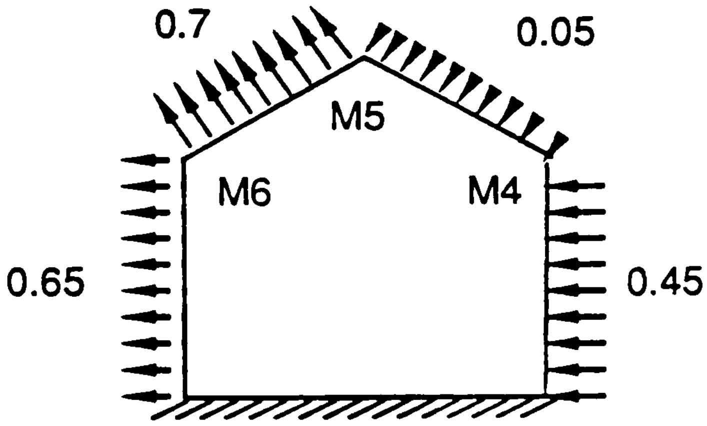


WR1

Figure 3.4 Wind Load Coefficients for Load Case I Applied from Left and Right (Table 3.4) for 7/12 (30.25^0) Frame, Interior Zone and Enclosed Case

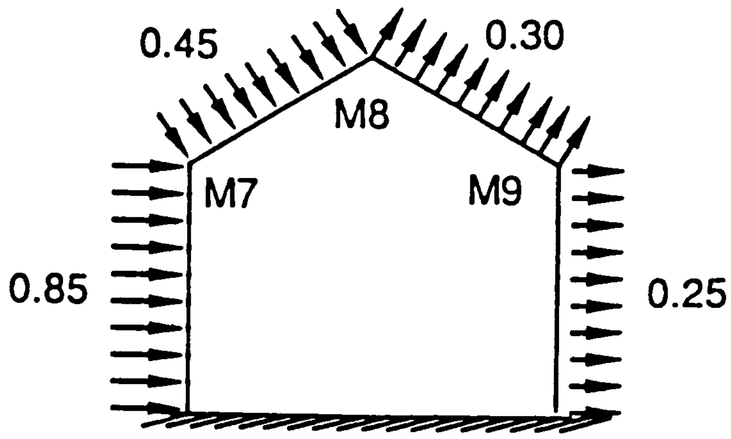


WL2 (Case II)

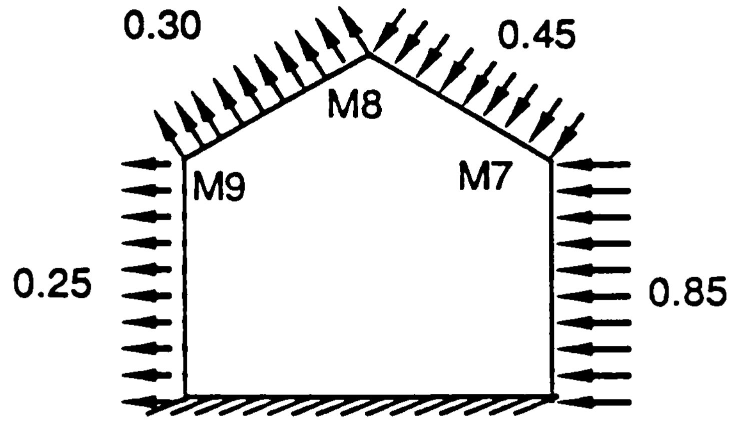


WR2

Figure 3.5 Wind Load Coefficients for Load Case II Applied from Left and Right (Table 3.4) for 7/12 (30.25^0) Frame, Interior Zone and Enclosed Case



WL3 (Case III)



WR3

Figure 3.6 Wind Load Coefficients for Load Case III Applied from Left and Right (Table 3.4) for 7/12 (30.25^0) Frame, Interior Zone and Enclosed Case

Industry practice is to design rafters in three segments of different depths measuring 15-25 feet each. For practical considerations and ease of fabricating the flange and web plate widths and thicknesses are kept constant in a particular segment. The web is tapered according to the moment diagram to optimize the weight. The flange and web plate widths and thicknesses are again optimized in the next segment. Optimization done in this fashion is restricted by the following:

1. It is not economically feasible to increase the number of segments beyond three or four, because of costs to connect the segments together.

2. Web plate thicknesses and flange plate widths are kept constant in a particular segment.

3. Starting depth of a segment is made equal to the ending depth of the previous segment to simplify connections.

Because of these restrictions, the ratio of actual to permissible stresses will be less than unity at some points in a segment. Main frames are designed according to AISC specifications. MBMA has also published a book (Lee et al. 1981) to help designers with the design procedures and formulas applicable to tapered rigid frames. Since this book has been published and endorsed by MBMA (Ellifritt 1981), the formulas and procedures as given in this book are used by metal building industry for designing tapered rigid frames.

3.5 Design of Rigid Frames Used in the Study

The rigid frames for two metal buildings, one with a low roof slope, and the other with a steep roof slope, are considered in this study. Roof slope is the most important parameter in determining the wind flow pattern and the wind pressures on the building envelope (Davenport et al. 1977, 1978, Sparks 1987). The pressure coefficients on the roof change significantly as the roof slope changes (ANSI 1982, MBMA 1986). MBMA 1986 specifies same pressure coefficients for the roof slope categories of $0\text{-}10^0$, $10^0\text{-}30^0$, and $30^0\text{-}45^0$. To evaluate the effects of roof slope, two buildings, one in the $0\text{-}10^0$ range and the other in the $30^0\text{-}45^0$ range, are considered. One building is designed for the roof angle of 2.4^0 (.5/12 roof slope, 0.5 vertical to 12 horizontal). The other building is designed for the roof angle of 30.25^0 (7/12 roof slope, 7 vertical to 12 horizontal). Metal building industry specifies the roof angle in terms of V/12, where V can be increased using the increment as low as 0.125. Specifying the roof angle in this manner helps erectors to establish and check the roof slope at site using conventional tools.

A typical single-span metal building system is comprised of gable frames designed for interior zone loading criteria, except for the two end frames that are designed to satisfy the end zone loading criteria (See Figure 3.1). If it is anticipated that a particular building will be expanded in future, its end frames are designed for the interior as well as the end zone loading criteria. Frames used in this study are designed for the interior zone only.

Metal buildings can be designed as enclosed or partially enclosed conditions of the building envelope. Enclosed condition means that there is a uniform distribution of openings in the building envelope, or doors and windows can be considered strong enough to resist the design wind pressures. Partially enclosed condition means that one of the wall has more than 5% area as open, or doors and windows are not strong enough to resist the design wind pressures. Performance of metal buildings in wind storms suggest that all metal buildings should be designed for wind loads on a partially enclosed building. Therefore, all failure modes and construction errors, except the breach of the building envelope, are studied on main frames designed for partially enclosed loads.

Only dead load + live load, and dead load + wind load combinations are considered in this study. Such load combinations usually govern the design in the regions frequently impacted by hurricanes and other extreme winds. A review of the hurricane prone regions listed in Appendix A reveals design wind speed in these areas lies in the range of 80-100 mph. Therefore, a design wind speed of 100 mph is considered. This study, therefore, may be applicable to the regions listed in Appendix A and other areas having similar design loads.

Two frames are designed for both the roof slopes for the interior zone of a partially enclosed 120 feet x 210 feet building. A 30-foot bay spacing and 100 mph design wind speed are used. Superimposed dead loads due to sheet metal, foam insulation and purlins are computed using Table 3.1. Dead loads of 3 lbs/ sq. ft. for .5/12 roof slope, and 3.5 lbs/ sq. ft. for 7/12 roof slope are used. A discussion with the local metal building manufacturers indicated that they also use 3-4 lbs/ sq ft. as the dead load. Self-weight of the frames is also included in the analysis and design. It may be noted that the roof dead load of the metal building systems is much lower than the other forms of roofs which may be as high 40 lbs/sq ft. The significant reduction in the roof dead load in metal buildings exacerbates the damaging effect of wind uplift. A design live load of 12 psf (Table 3.2) is used.

The frames are designed using the DESIGN program (Synercom 1988). This program implements the AISC design equations given in Appendix C. The method for analyzing the frame is explained in section 3.4. The frames have been designed using commercially available plate stock of 50 ksi yield strength. These frames have also been optimized to reduce the weight as far as practicable.

3.5.1 .5/12 Roof Slope Building

Figure 3.7 shows the important structural details of the .5/12 frame. The frame has one length segment for the column and three length segments for the rafter. Flange and web plate sizes as well as depths at various sections are indicated. Outer flanges are on the outer side of the column and rafter and they support girts and purlins. Four girts are attached to one column and fourteen purlins are attached to one rafter. Figure 3.8 shows the moment profile on the left rafter for the DL+LL and all four loading conditions for DL+WL case. Maximum design moment profile has also been indicated. The frame is designed and optimized using this moment profile. Figure 3.9 shows the moment profile on the left column for the DL+LL and all the four loadings conditions for DL+WL case. Total weight of the .5/12 frame (two rafters and two columns) is 6488 pounds.

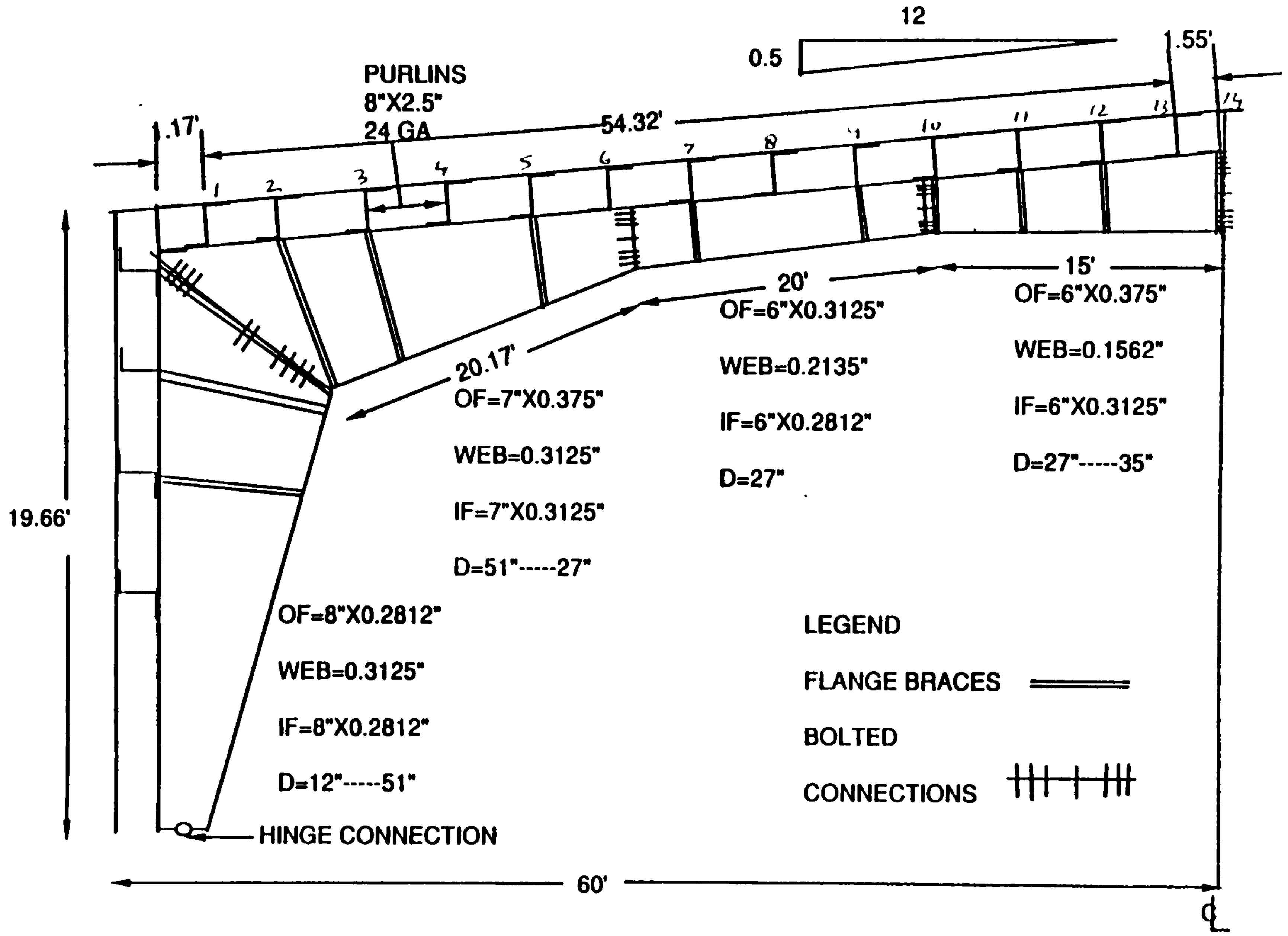


Figure 3.7 Structural Design Details of the .5/12 Roof Slope Frame Designed for 100 mph Wind, 120 ft Span and Interior Zone

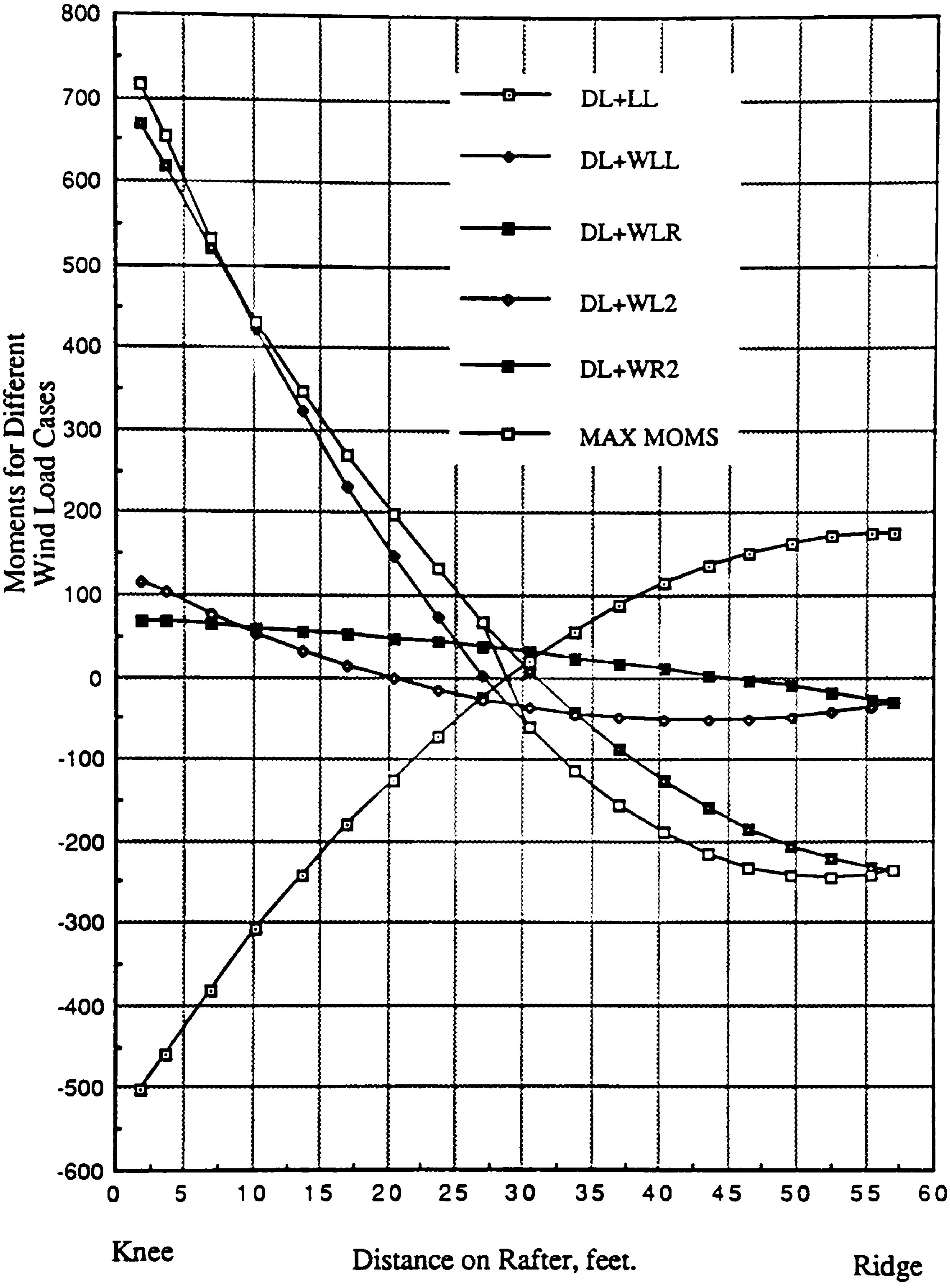


Figure 3.8 Plot of Moments for Different
Wind Load Cases on the Rafter.
(Span 120 ft., Wind Speed=100 mph
Roof Slope=.5/12, Int Zone, Partially Enclosed)

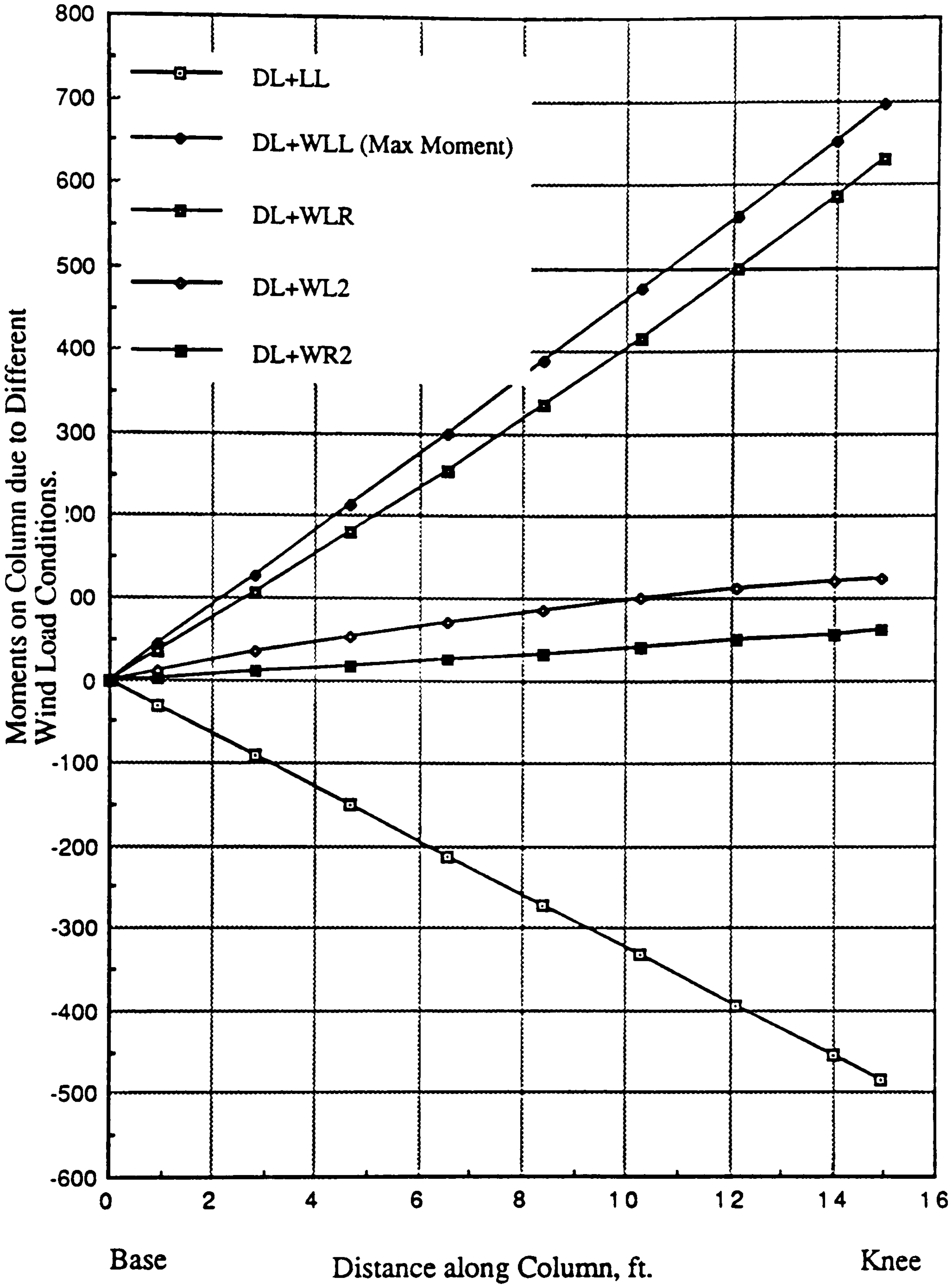


Figure 3.9 Plot of Bending Moments on Column due to Different Wind Load Combinations. (Span 120 ft., Wind Speed=100 mph, Roof Slope .5/12, Int. Zone, Partially Enclosed)

3.5.2 7/12 Roof Slope Building

Figure 3.10 shows the important structural details of 7/12 frame. Figure 3.11 shows the moment profile on the left rafter for the DL+LL and all the six loadings conditions for DL+WL case. Maximum design moment profile has also been indicated. The frame is designed and optimized using this moment profile. Figure 3.12 shows the moment profile on the left column for the DL+LL and all the six loadings conditions for DL+WL case. Total weight of the 7/12 frame is 7479 pounds.

Effects of different failure modes are studied by computing the changes in the design moment profile (maximum of DL+LL and DL+WL) and the unity checks along the rafters and columns. All the graphs presented subsequently are with reference to this design moment profile and the corresponding AISC unity checks. AISC unity checks are according to the formulas referred in Appendix C.

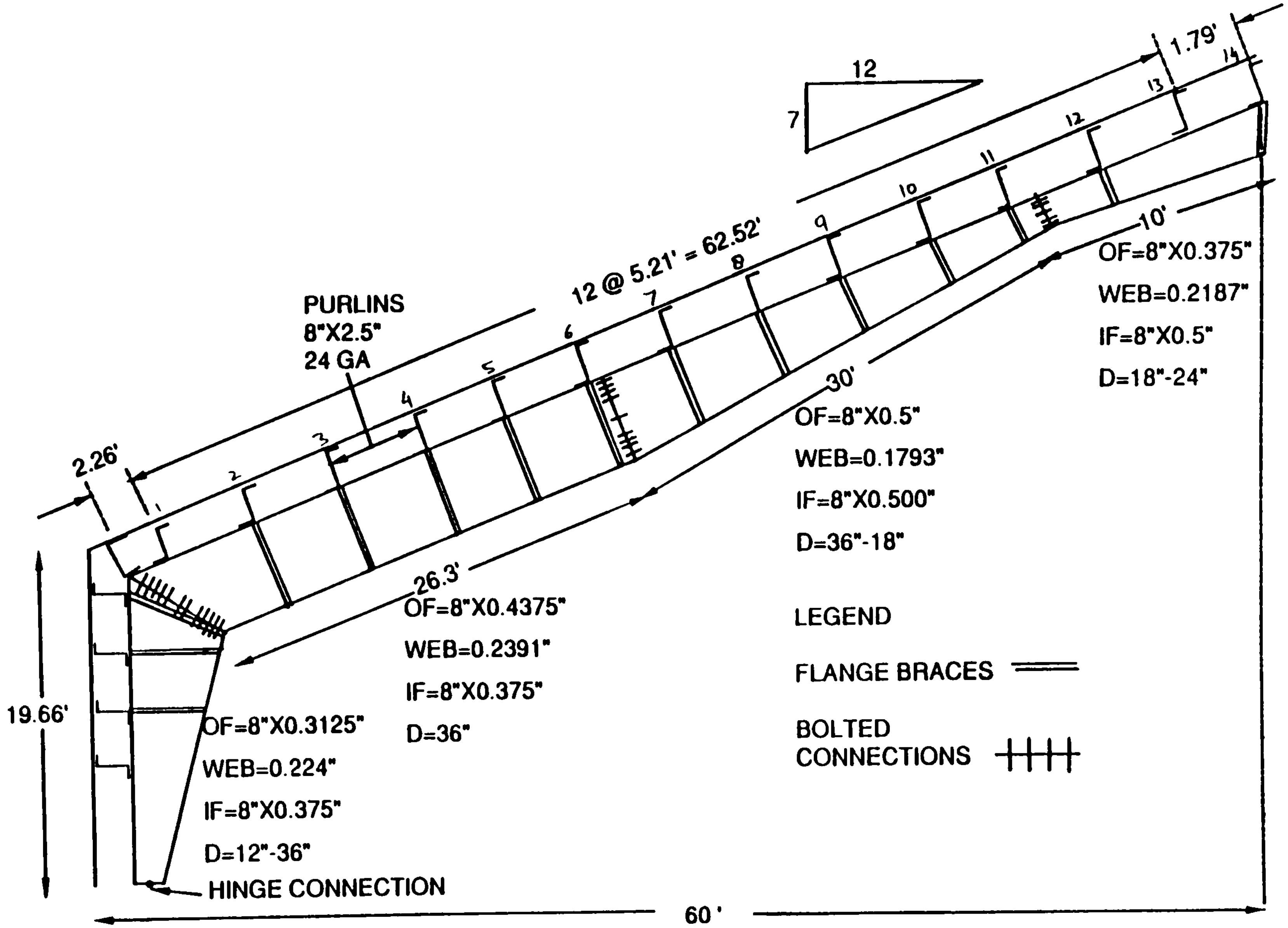


Figure 3.10 Structural Design Details of the 7/12 Roof Slope Frame Designed for 100 mph Wind, 120 ft Span and Interior Zone

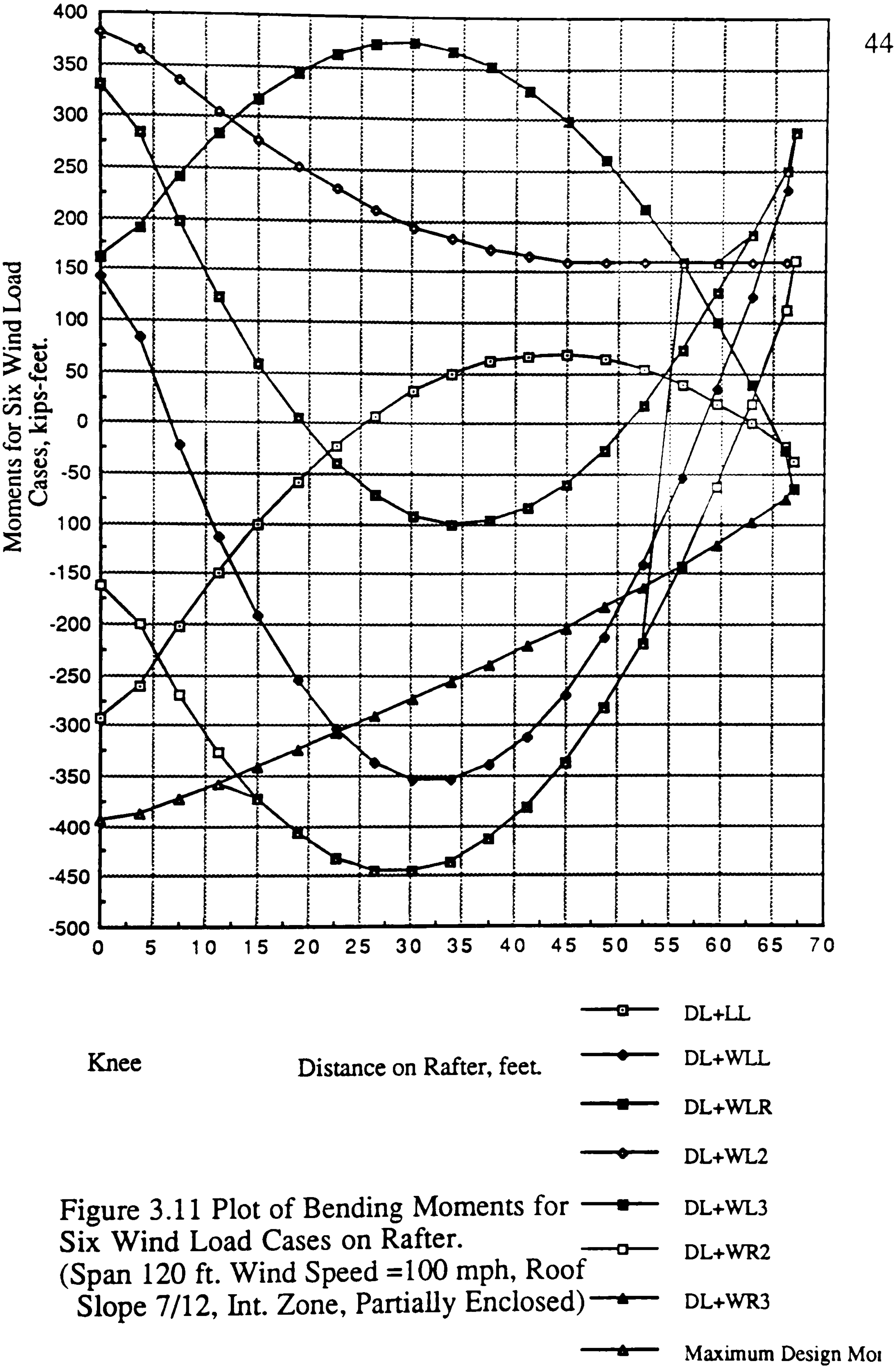


Figure 3.11 Plot of Bending Moments for Six Wind Load Cases on Rafter.
(Span 120 ft. Wind Speed =100 mph, Roof Slope 7/12, Int. Zone, Partially Enclosed)

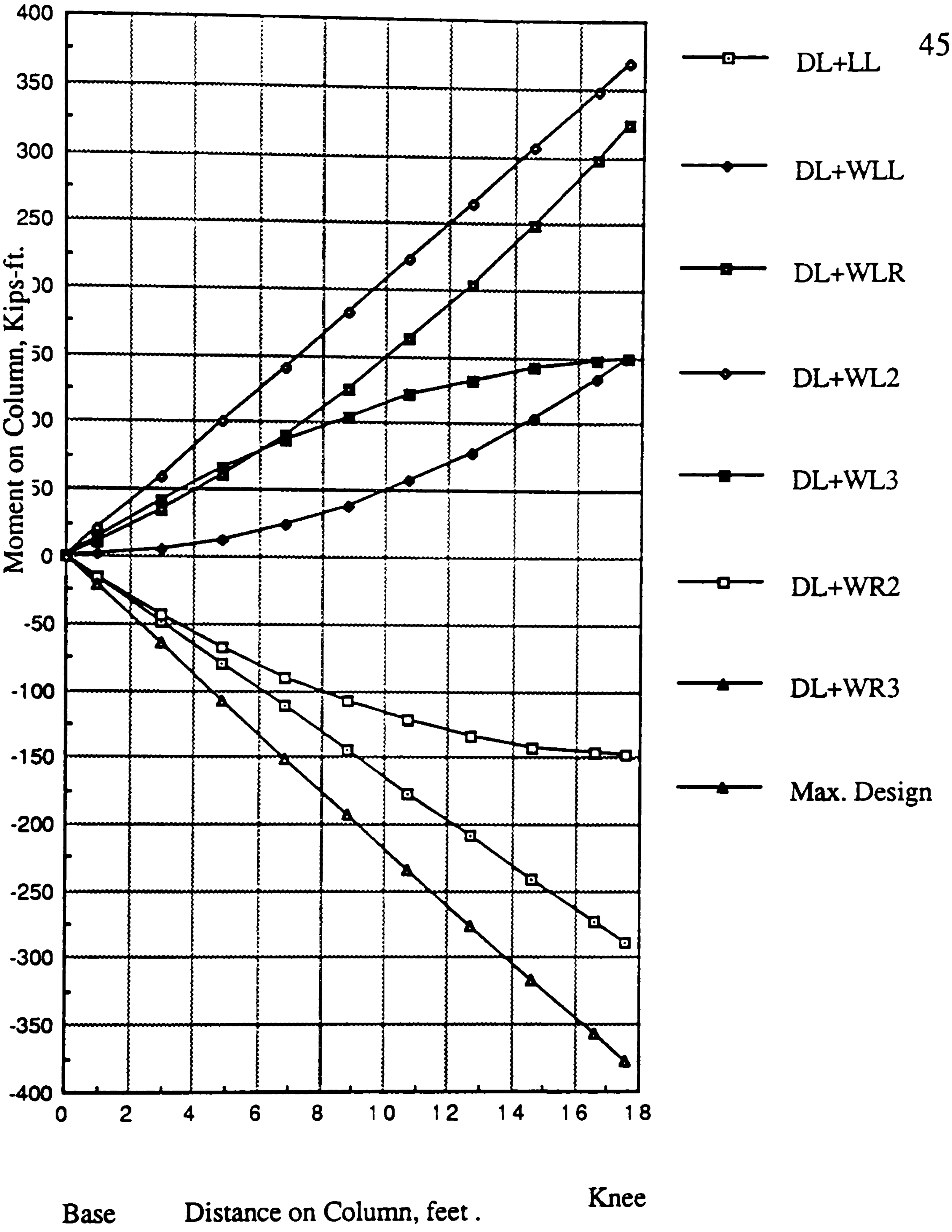


Figure 3.12 Plots of Bending Moment on Column for Six Wind Load Cases.
(Span 120 ft. Wind Speed=100 mph, Roof Slope 7/12, Int zone, Partially Enclosed)

CHAPTER IV
ANALYSIS AND EVALUATION

4.1 Breach of Building Envelope

Breach of building envelope is the most prevalent reason for damage to metal buildings in extreme winds. A breach in the building envelope in a windward wall results in higher internal pressures. If sufficient size and distribution of openings occur, the building becomes partially enclosed. For design purposes a different set of pressure coefficients must be used. Effects of breach of building envelope are evaluated, by computing the changes in the design moment, and the increases in axial and bending stresses resulting from wind pressure on a partially enclosed building. Comparisons are made in terms of the unity check of AISC along the column and the rafter. Increases in the horizontal and vertical reactions on base connection are also computed.

Breach can occur due to stripping of sheet metal panels, failure of doors and windows, or by the perforations of wind-borne missiles in a wind storm. MBMA 1986 criteria specifies that buildings having 5% openings in the dominant wall shall be considered as partially enclosed buildings. However, recent research indicates that failures of doors comprising 1-5% of the area of a windward wall may be sufficient to produce a significant increase in the internal pressure (Vickery et al. 1984).

If a building is designed as enclosed, then the doors, windows and other cladding should be able to resist design pressures and impact effects.

However, the designer does not normally specify the doors and windows and a lack of coordination between designer and contractor may result in doors and windows that cannot withstand the design wind pressure. The cladding on ordinary metal buildings will not resist perforations of missiles such as 2 x 4 planks. An argument could be made at that proper interpretation of industry standard requires that a high percentage of metal buildings be designed as "partially enclosed buildings" although this is not being done (Perry 1989).

The industry standard specifies wind loads for enclosed and partially enclosed buildings. However, the standard does not permit designing for partially enclosed conditions for buildings falling in the enclosed category. Designers feel protected by designing according to the legal standards, if the building gets damaged for any reason whatsoever, the designer can be blamed for not designing according to the legal standard. Further, no studies have been undertaken to prove that partially enclosed designs are also safe designs for enclosed conditions for all building shapes, geometries, and wind loads. No studies have been undertaken to collect data on cost increases for designing frames for partially enclosed conditions or frames suitable for both conditions. And lastly, it should be understood that the metal building industry is a very competitive industry. Research findings that lead to increases in costs will not be accepted by manufacturers until they become part of the design code.

To evaluate the effects of breach in the building envelope, main frames are designed for the two roof slopes for the enclosed condition. Frames are then analyzed with wind loads for the partially enclosed conditions. All loading cases for the partially enclosed condition are considered. A design wind speed of 100 mph is used for both the enclosed and the partially enclosed conditions. Changes in the design moment profile (max of DL+LL and DL+WL) and the unity check are computed along the lengths of the rafter and column. Changes in the vertical and horizontal reactions on base connection are also computed. Findings are given below.

4.1.1 Low-Slope Building

4.1.1.1 Effect on Rafter

Dramatic changes in the moment profile along the rafter are observed. Figure 4.1 shows the moments for the DL+WL case for the enclosed and the partially enclosed conditions. A positive moment indicates compression in the outer flange all along the frame. The most important finding is reversal of the sign of the moment along the rafter. In the rafter up to 10 feet from the knee, the DL+LL governs the design (Figure 3.8). Therefore, the increases in the wind induced moment and the unity check are not critical at the knee (Figures 4.1 and 4.2). At the ridge reversal in the sign of moment and the increase in the unity check suggests the need for bracing of inner flange (Figures 4.1 and 4.2). At a section 43.5 feet from knee where DL+WLL loading condition governed the design an 8% increase in the unity check is observed. Substantial increases in stresses are observed at many other sections of the rafter also (Figure 4.2).

4.1.1.2 Effect on Column

For the column designed for the enclosed condition DL+LL governed the design. Partially enclosed loads gave moments in the opposite sense of the enclosed loads (Figure 4.3). This indicated the need for adequate

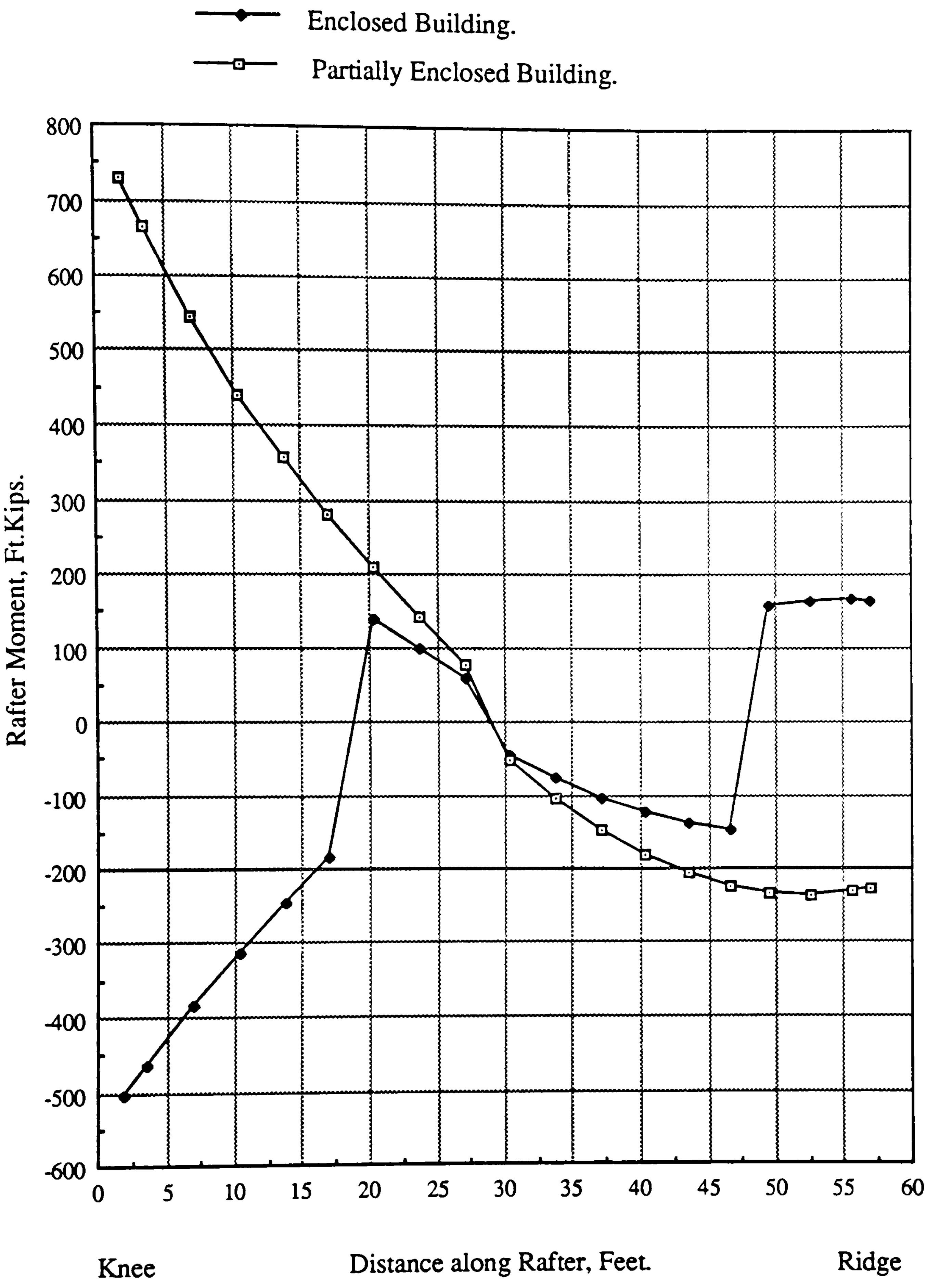


Figure 4.1 Rafter Moments for Low-Slope Frame.
(Span 120 ft, Wind Speed=100 mph, Roof Slope .5/12,
Int Zone, Frame Designed for Enclosed Case)

Figure 4.2 Unity Check for Low-Slope Frame.
(Span 120 ft., Wind Speed=100 mph, Roof Slope .5/12,
Int.Zone, Frame Designed for Enclosed Building)

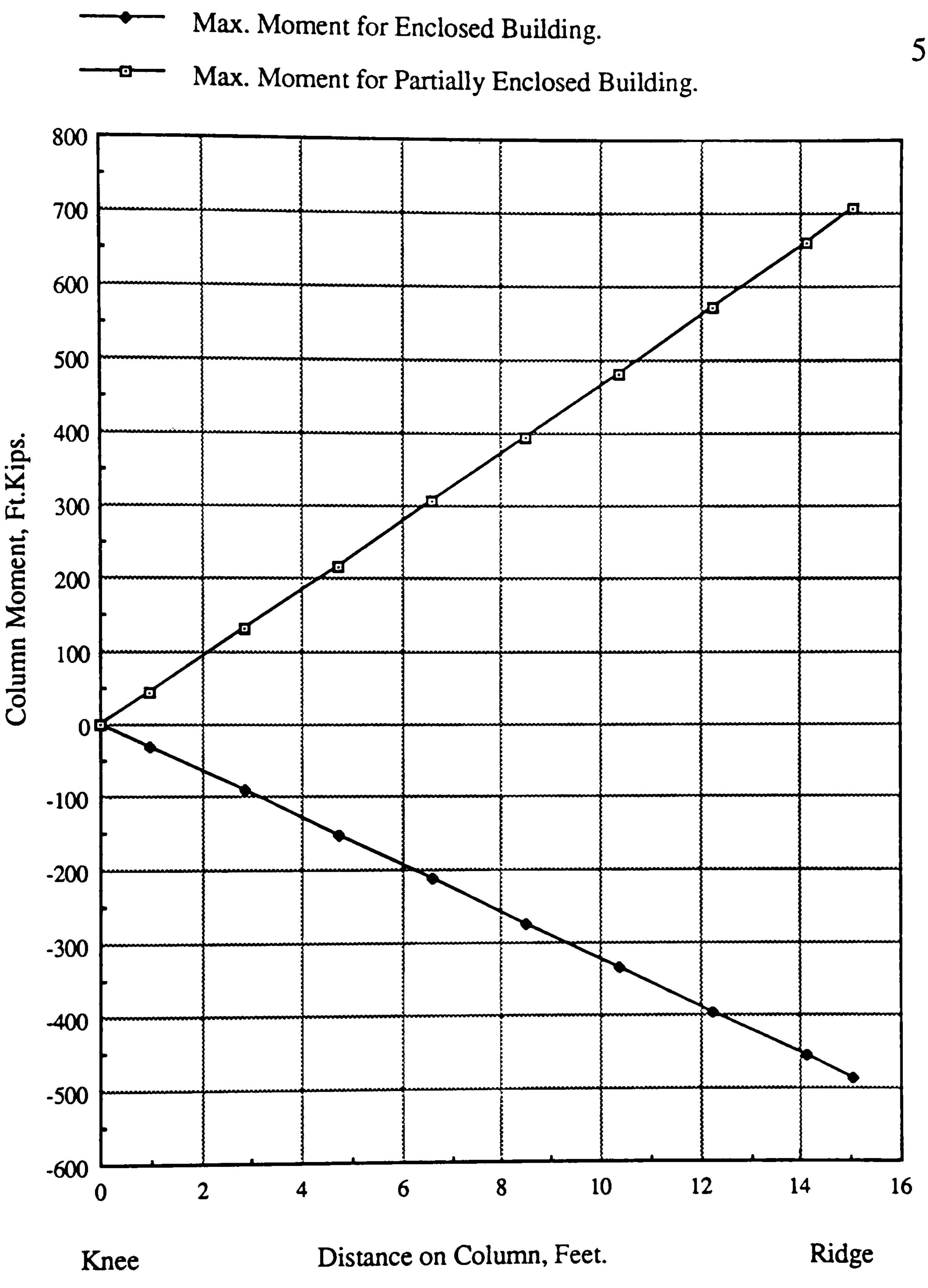


Figure 4.3 Column Moments for Low-Slope Frame. (Span 120 ft., Wind Speed=100 mph, Roof Slope .5/12, Int. Zone, Frame Designed for Enclosed Case)

bracing of the outer flange of the column. Outer flange may not otherwise have been braced for the enclosed design. A 3% increase in the unity check is also observed at many points on the column (Figure 4.4).

Further it is found that the uplift at the column base increased from 28.3 kips to 44.2 kips in going from enclosed to partially enclosed loading. The horizontal reaction on the column base increased from 36 to 51.3 kips. These increases are sufficient to bring the induced stresses very close to the yield stress.

4.1.2 High-Slope Building

4.1.2.1 Effect on Rafter

Partially enclosed loading increased the magnitude of moments all along the rafter (Figure 4.5). The stresses exceed allowable values between 26 to 51 feet and from 58 feet to the ridge (Figure 4.6). The highest stresses occur 40 feet from the knee and at the ridge. The moment at 40 feet from knee changed from -314 to -381 ft-kips, while the corresponding unity check went from 0.915 to 1.19. The ridge moment increased from 186 to 287 ft-kips with a corresponding increase in unity check from 0.94 to 1.38. Unlike the low-slope frame the sense of the moments did not change (Figures 4.5 and 4.6).

4.1.2.2 Effect on Column

The moments along the column increased marginally (Figure 4.7). Only 2-3% increase in the unity check is observed (Figure 4.8). However, the effects on the base connection are significant. Design uplift for the base

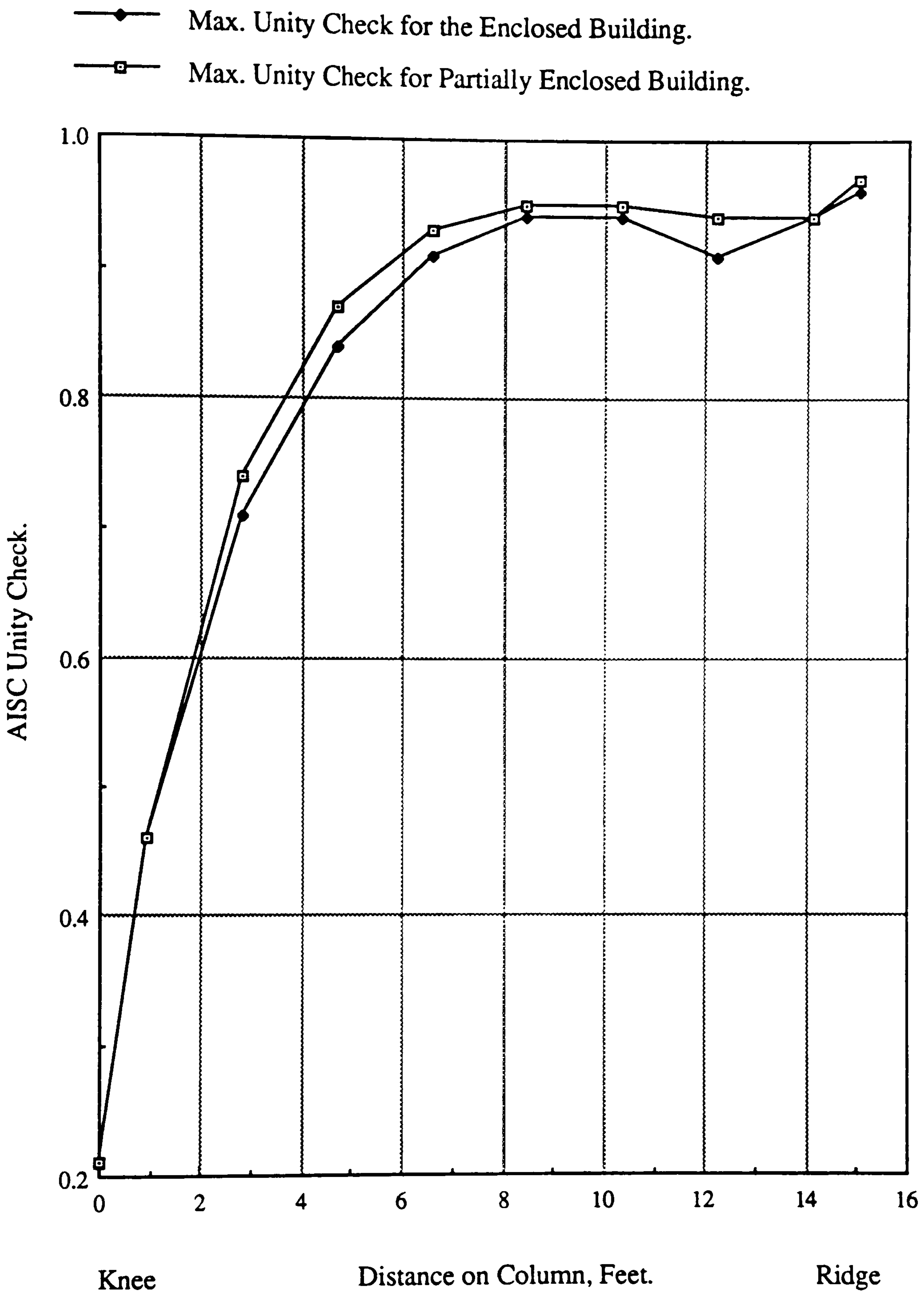


Figure 4.4 Unity Check for Low-Slope Frame.
(Span 120 ft., Wind Speed=100 mph, Roof Slope .5/12,
Int. Zone, Frame Designed for Enclosed Case)

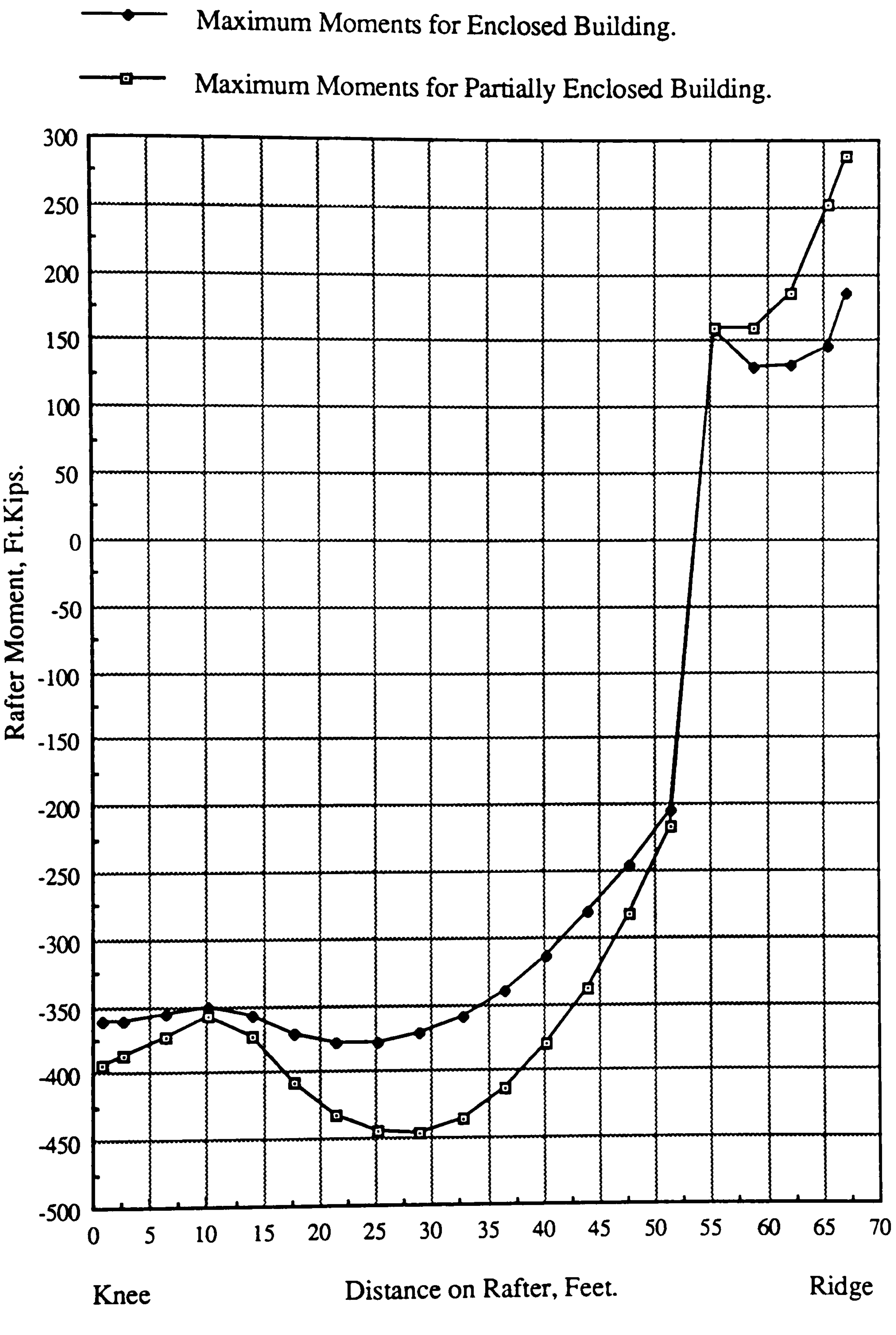


Figure 4.5 Rafter Moments for the High-Slope Frame. (Span 120 ft., Wind Speed=100 mph, Roof Slope 7/12, Int. Zone, Frame Designed for Enclosed Case)

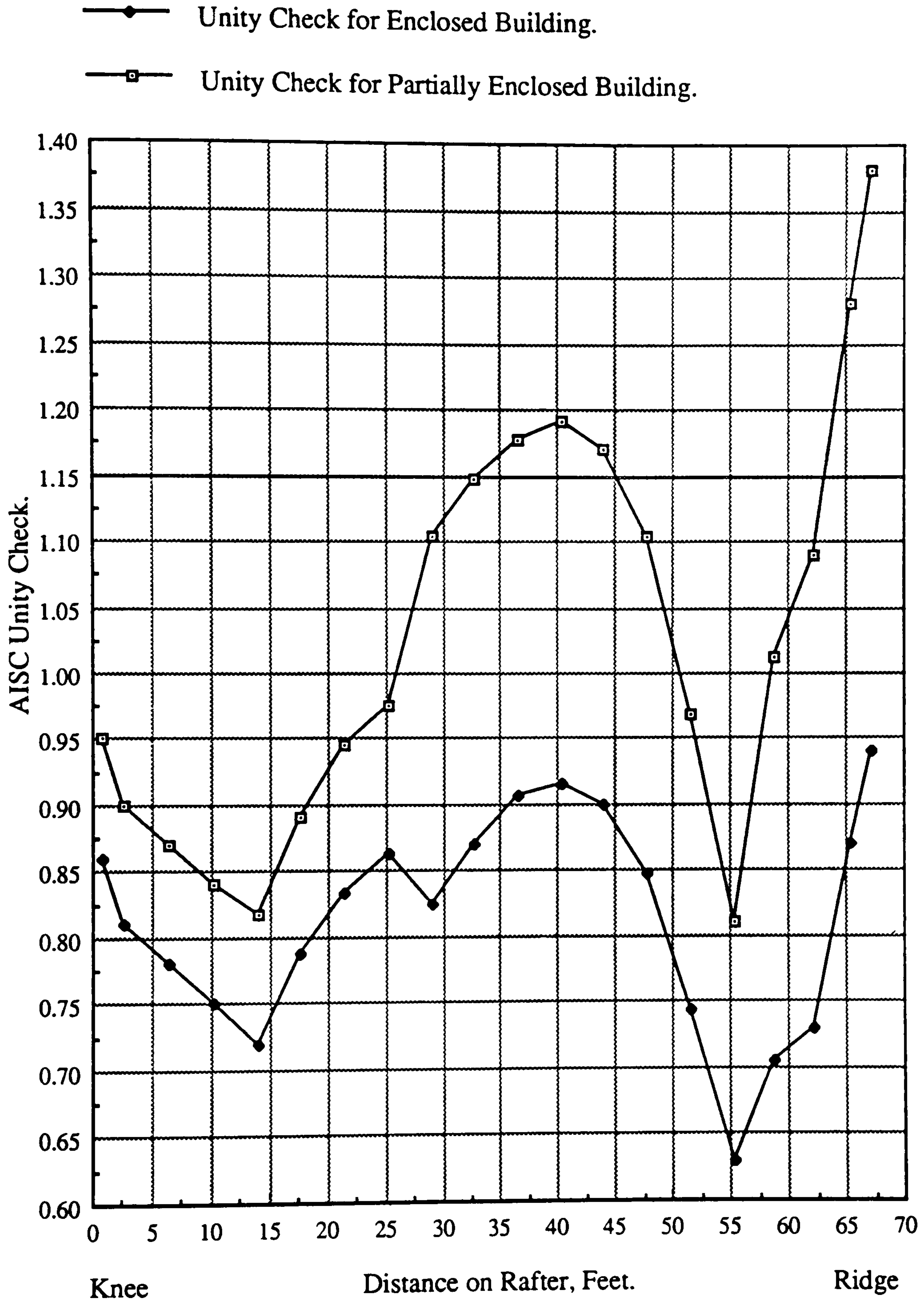


Figure 4.6 Unity Check for High-Slope Frame.
(Span 120 ft., Wind Speed=100 mph, Roof Slope 7/12,
Int. Zone, Frame Designed for Enclosed Case)

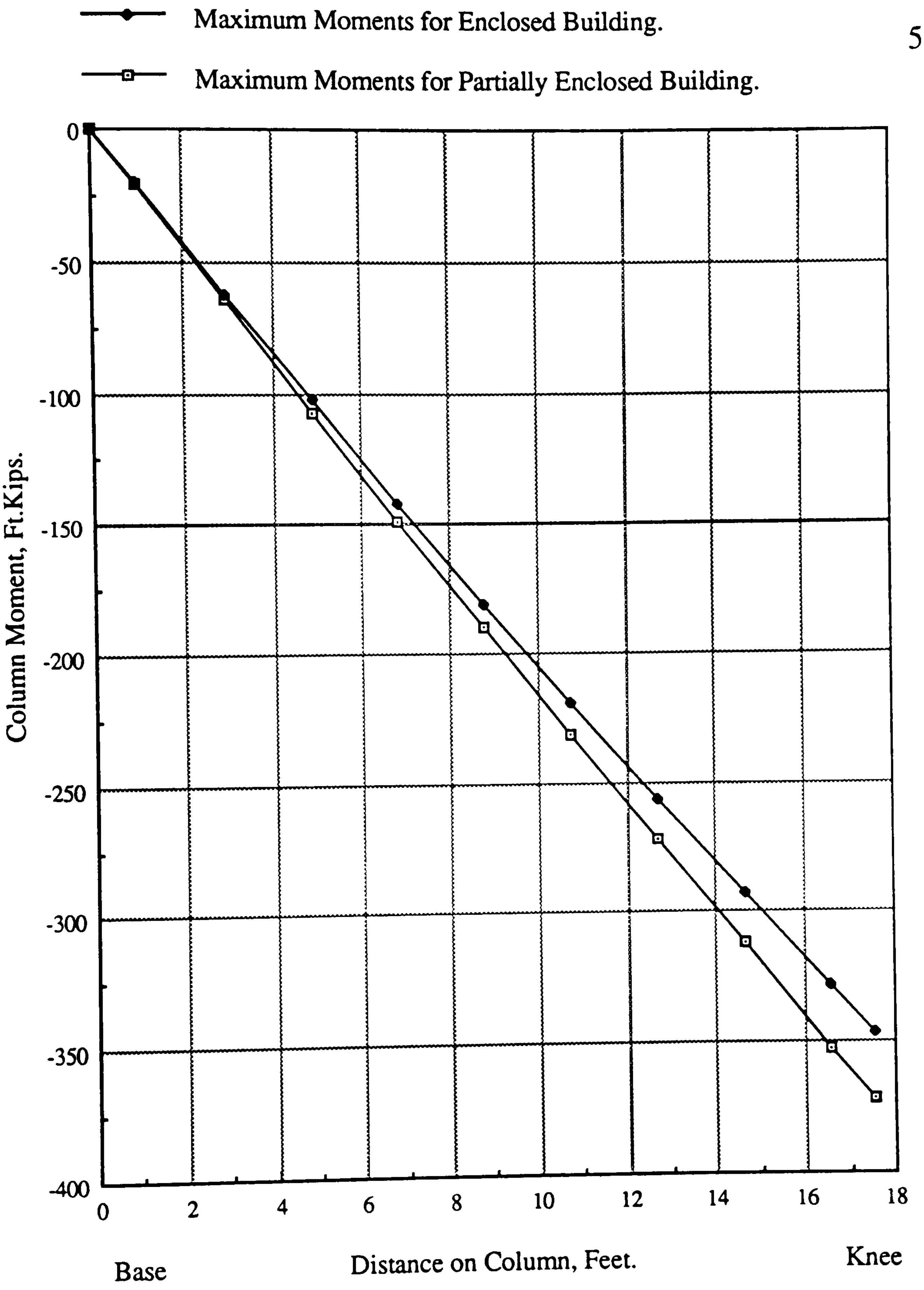


Figure 4.7 Column Moments for High-Slope Frame. (Span 120 ft., Wind Speed=100 mph, Roof Slope 7/12, Int. Zone, Frame Designed for Enclosed Case)

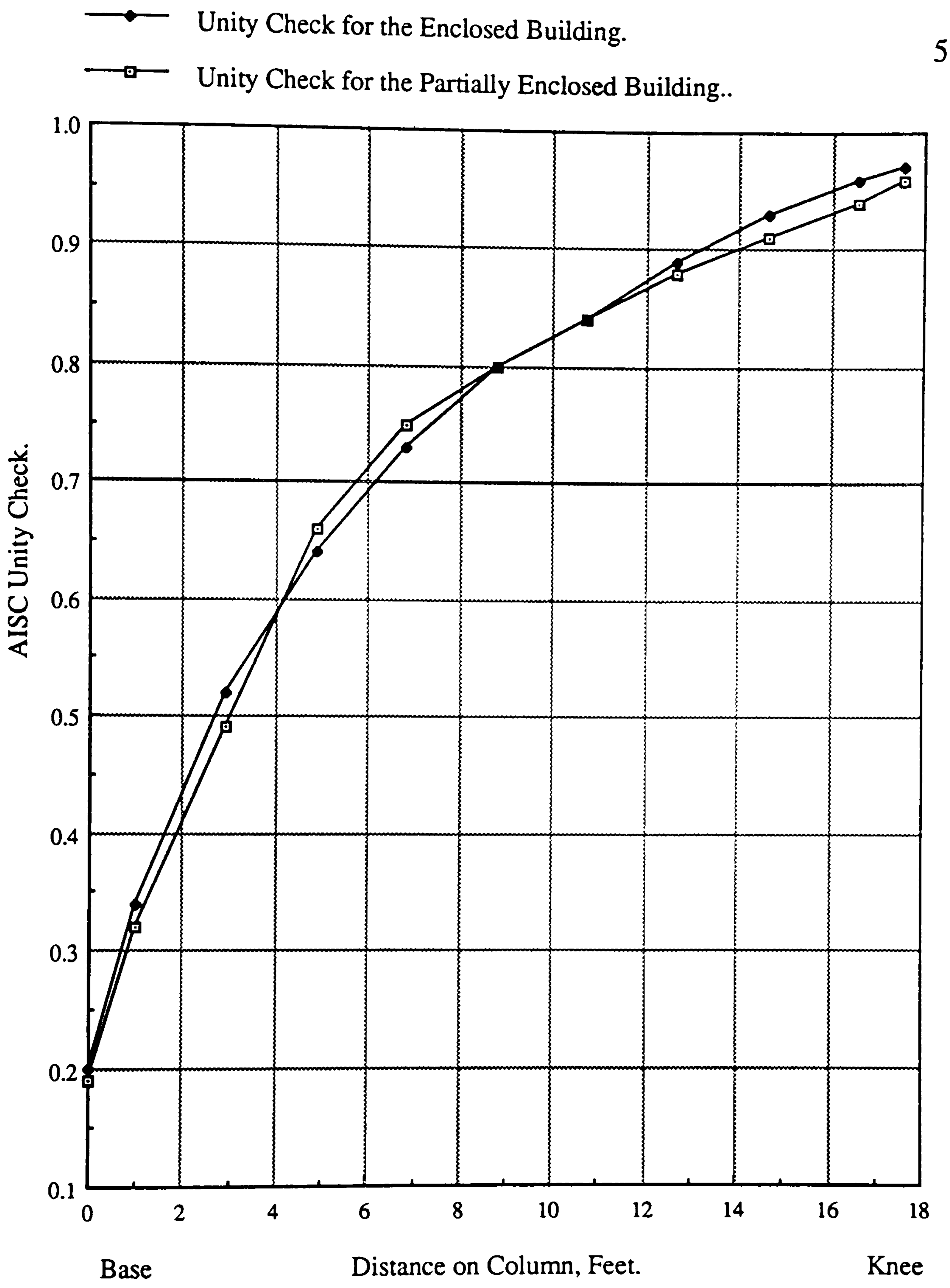


Figure 4.8 Unity Check for High-Slope Frame.
(Span 120 ft., Wind Speed=100mph, Roof Slope 7/12,
Int. Zone, Frame Designed for Enclosed Case)

connection increased from 31 kips to 50 kips. Design horizontal reaction also increased from 22.2 kips to 23.2 kips.

4.1.3 Weight Increase

Main frames are designed for enclosed and partially enclosed conditions for both the roof slopes. The low-slope frame weighs 6121 pounds for the enclosed case and 6488 pounds for the partially enclosed case. For the high-slope frame, the weights are 6289 and 7479 pounds, respectively. The overall construction cost for a 120 feet wide metal building system may be assumed as $12 per square foot. The cost of material and fabrication for the structural frame is approximately $0.65 per pound (Star Building Systems 1990). It is found that to design a frame to resist partially enclosed conditions the cost increase will be only $0.07 per square foot for the low-slope frame and $0.21 per square foot for the high-slope frame. Therefore for a low-slope building a project cost increase of 0.6% and for a high-slope building a project cost increase of 1.75% will provide frames capable of resisting partially enclosed loadings.

4.2 Omitted Bracing of Compression Flanges

The columns and rafters of the main frame are designed as members of a plane frame. These frames have great strength in the vertical direction but will undergo lateral-torsional buckling, if the compression flange is not adequately braced. The flange bracing members are installed at 4-5 feet intervals along the columns and rafters and are bolted to the purlins or girts. The lateral restraint provided by purlins and flange bracing is relied upon to keep the main frame in the vertical plane.

Sometimes flange bracing is not installed correctly by the contractor or is omitted altogether. Flange bracing normally is installed after the main frames, purlins and girts have been bolted in place. Since the flange bracing is not required for the frames to carry gravity loads, they may be omitted because the erector does not fully understand their purpose (Perry et al. 1989).

To evaluate the effects of the omitted bracing of the compression flanges, it is assumed in this study, that the frame does not have the lateral restraint of the inner compression flange at two adjacent points in the region of maximum design negative moment. The allowable bending stress of the unbraced frame segment is computed using AISC procedures or the design charts for the tapered members by Lee et al. (1981). Another built-up frame section having bracing like in the original frame is replaced in that section. The built-up frame has a smaller compression flange (in width as well as thickness) but has the same bending capacity as that of the laterally unsupported segment. The modified frames are then analyzed using the same loads as used in the original frames. Findings are given below.

4.2.1 Low-Slope Building

Design calculations lead to three inner flange plate size of 7"x.0.3125", 6"x.0.2812" and 6"x.0.3125" to be braced by 9 flange braces as shown in the Figure 3.9. Braces at purlins 10 and 11 are removed and the section between purlins 9 and 12 is modified by using the bottom flange 4" x 0.125" in a segment 9 feet long. This modified section is arrived at after several trials and gives the same compressive stress value with the use of

AISC design formulas for web tapered sections and the improved formulas (Lee et al. 1981).

Figures 4.9 and 4.10 show the redistributed moment and the unity check along the rafter, respectively. The moment at a section located at 24 feet from the knee increased from 132 to 146 ft-kips (Figure 4.9). The unity check at this section increased from 0.80 to 0.87 (Figure 4.10). The strongest effect is noticed in the replaced section, where moment changed by less than 10% but the unity check changed by 200%. Figures 4.11 and 4.12 show the redistributed moment and the unity check along the column, respectively. The unity check at a section 10.2 feet from the base the increased from 0.90 to 0.98 (Figure 4.12). These computations clearly indicate the importance of lateral bracing. Loss of even one bracing for the compression flange will substantially reduce the bending resistance and localized buckling failure may start.

4.2.2 High-Slope Building

A similar approach for simulating the omission of flange braces as used in low-slope is used for the high-slope frame. Design calculations lead to three inner flange plate size of 8"x.0.375", 8"x.0.5" and 8"x.0.5" to be braced by 12 flange braces as shown in the Figure 3.10. Braces at purlins 6 and 7 are removed and the section between purlins 5 and 8 is modified by using the bottom flange of size 4"x 0.3125" in a length segment of 10 feet. Localized effects are noted in the segment where the modified section is replaced. The unity check in this region increased by more than a factor of two (Figure 4.13). In addition to the localized effects at a section 63 feet

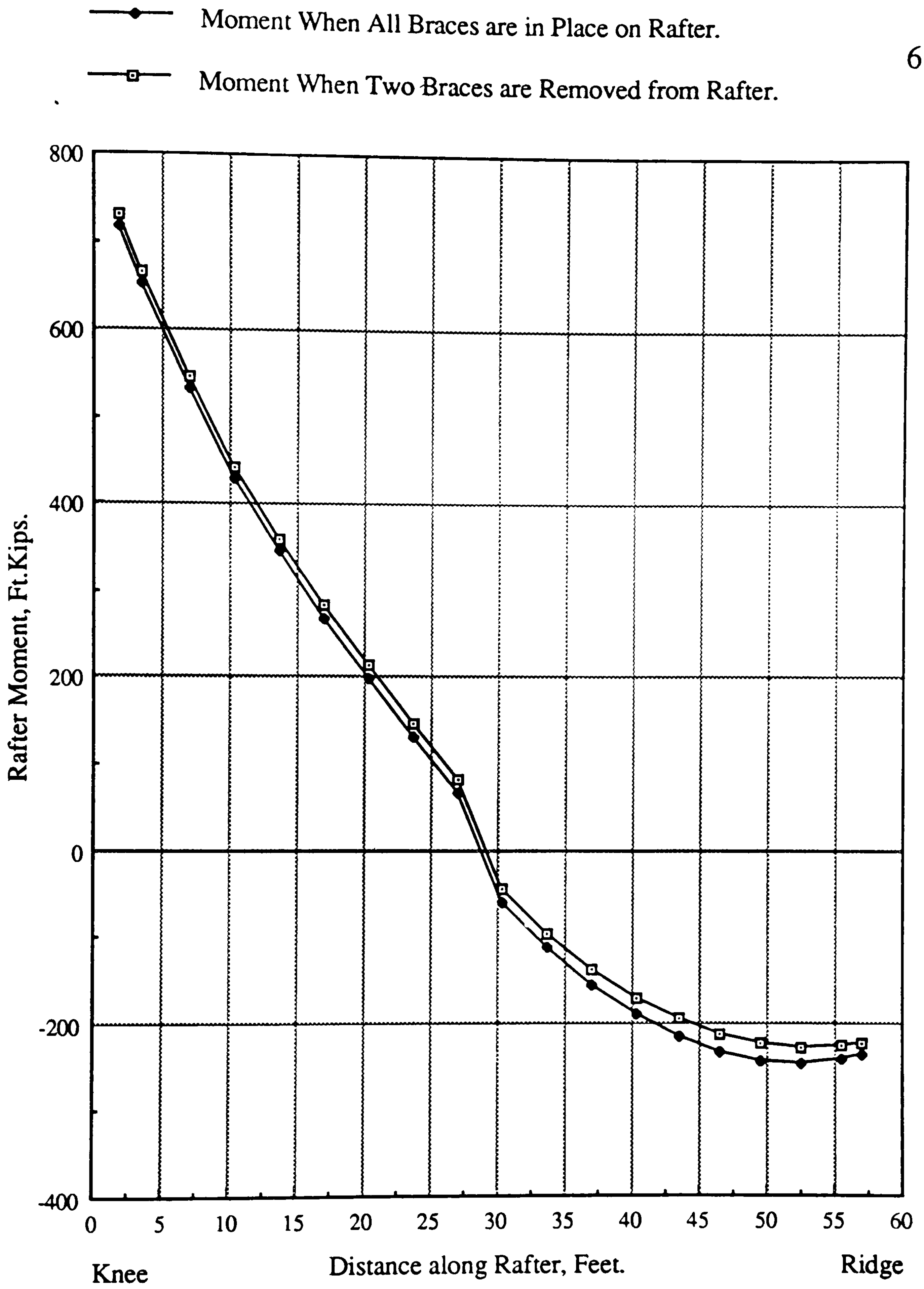


Figure 4.9 Rafter Moments for Low-Slope Frame.
(Span 120 ft, Wind Speed=100 mph, Roof
Slope .5/12, Int. Zone, Partially Enclosed)

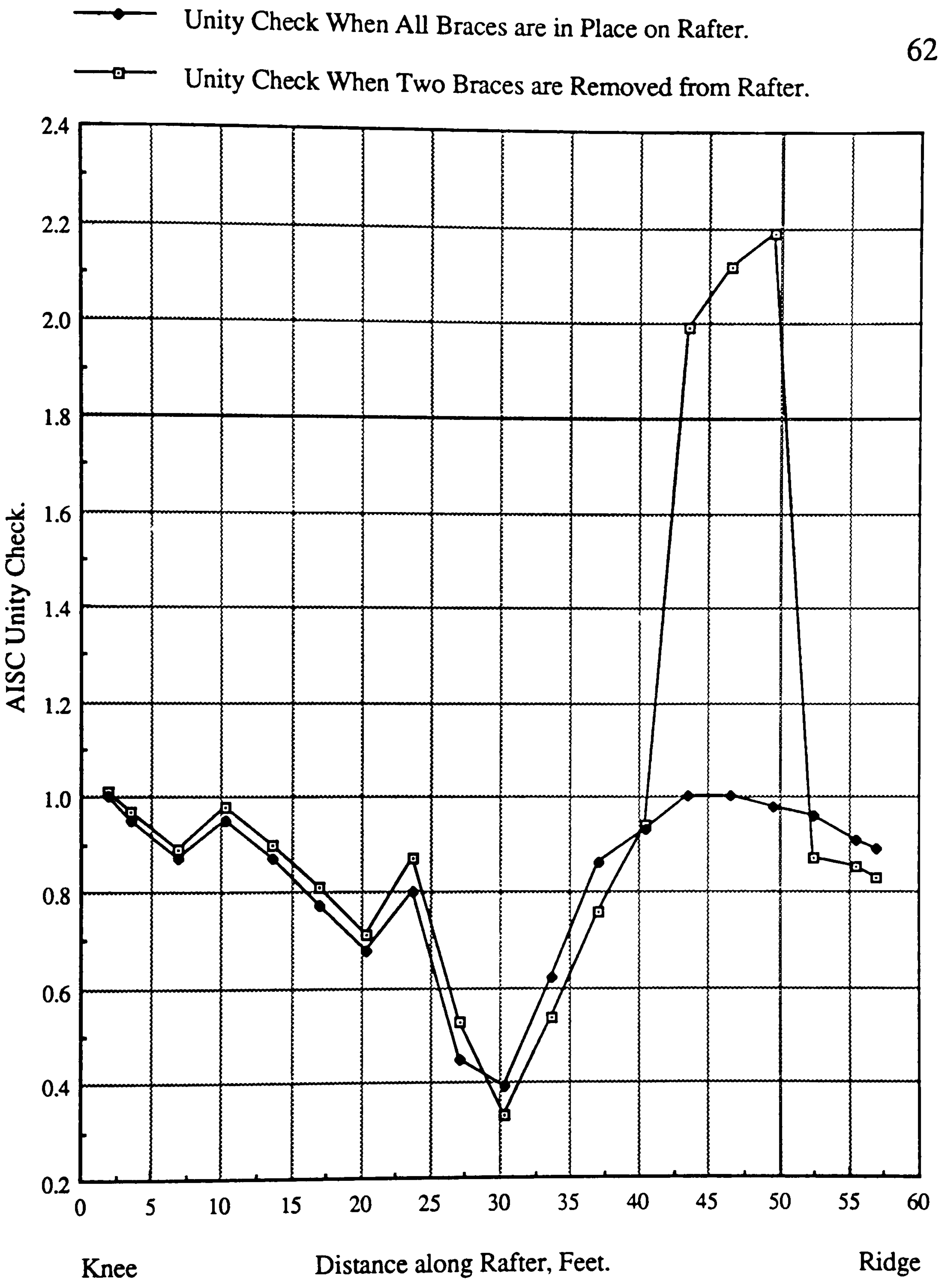


Figure 4.10 Unity Check for Low-Slope Frame. (Span 120 ft, Wind Speed=100 mph, Roof Slope .5/12, Int. Zone, Partially Enclosed)

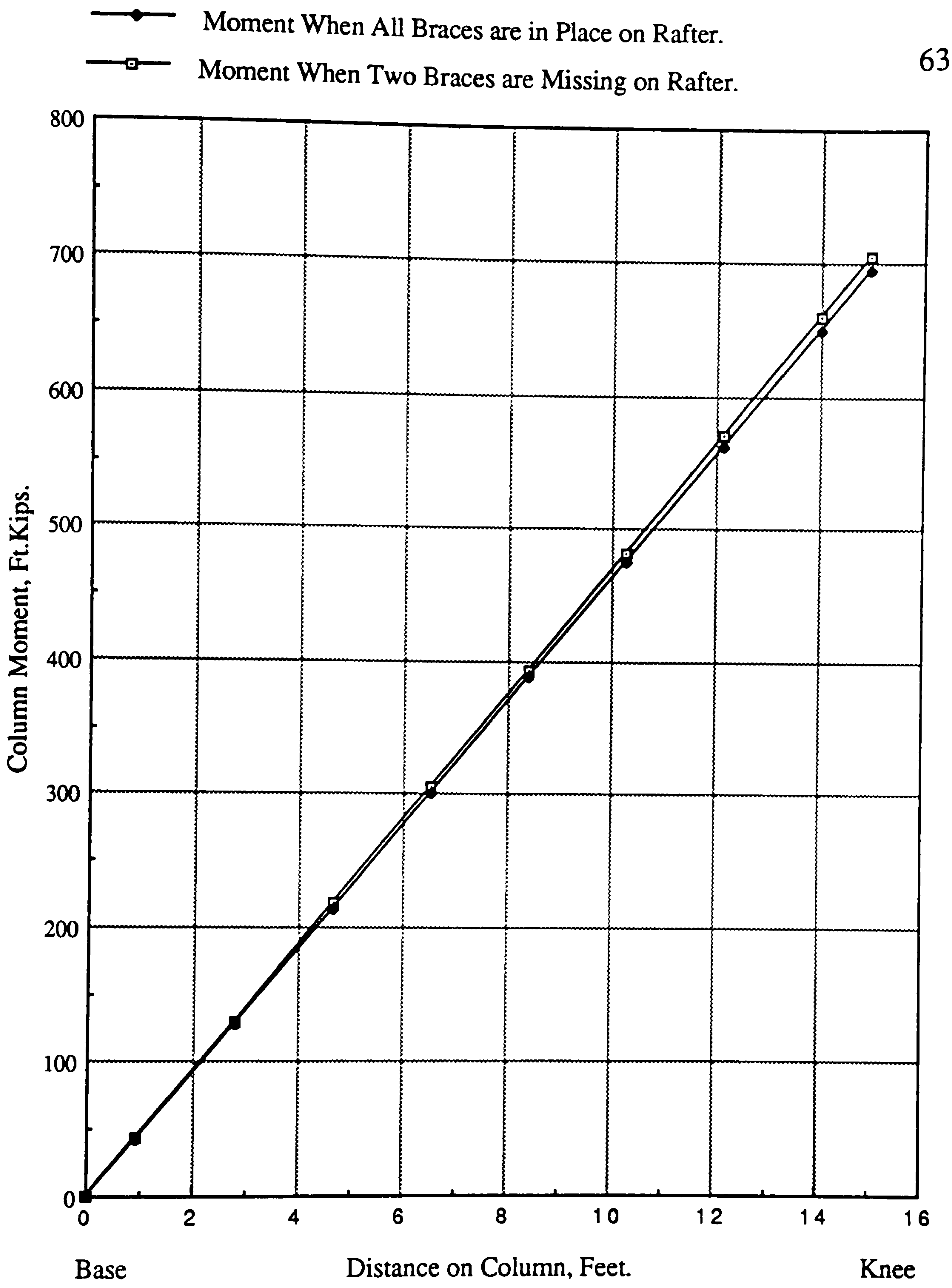


Figure 4.11 Column Moments for Low-Slope Frame.
(Span 120 ft., Wind Speed=100 mph, Roof
Slope .5/12, Int. Zone, Partially Enclosed)

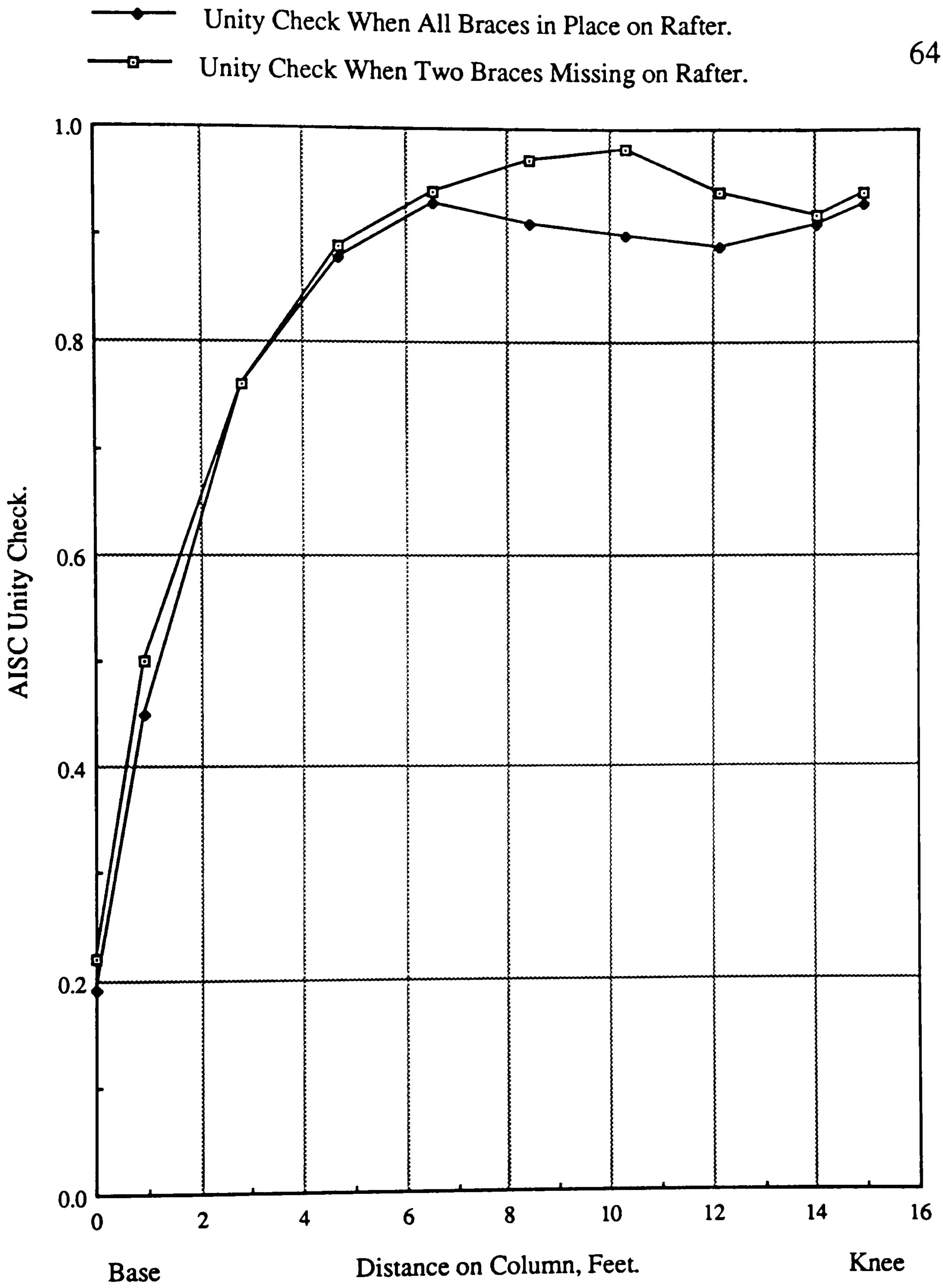


Figure 4.12 Unity Check for Low-slope Frame. (Span 120 ft, Wind Speed=100 mph, Roof Slope .5/12, Int Zone, Partially Enclosed)

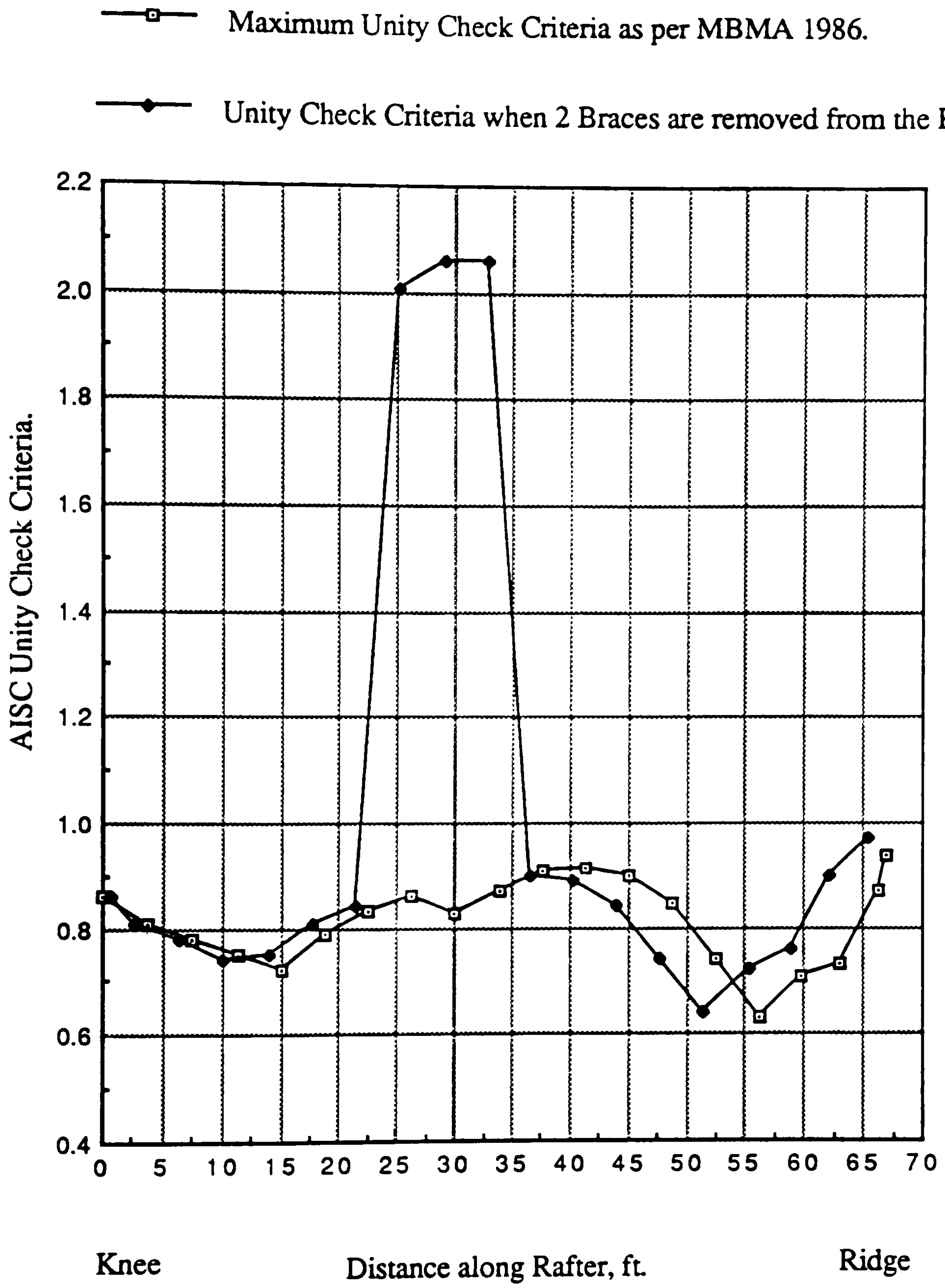


Figure 4.13 Unity Check for High-Slope Frame. (Span=120 ft., Wind Speed=100 mph, Roof Slope 7/12 Int. Zone, Partially Enclosed)

from the knee, the unity check increased from 0.73 to 0.91. No effects are noted in the moment profile or the unity check of the column.

4.3 Purlin Anchorage Failures

Wind uplift forces that act on roof sheet metal are transferred to the purlins through the tapping screws. Tapping screws are typically placed at 1-foot intervals along the length of the purlin (Buettner et al. 1990). Wind uplift forces on the purlins are transferred to the main framing through simple bolted connections between the purlins and the top flange of the rafter (Buettner et al. 1990). Purlins are anchored to rafters using A307 bolts. Purlin anchorage failures can occur with the sheet metal still attached to the purlins. Purlin anchorage failures may occur due to mis-punched holes, missing and incorrect size of bolts, or fatigue of sheet metal around the bolted connections .

The purlin connections are designed using pressure coefficients for parts and portions (MBMA 1986). The parts and portion coefficients in the edge strips and corners are 70 to 100% higher than the main wind-force coefficients (MBMA 1986). To simplify the drilling and bolting processes, common industry practice is to use the same size bolts in all purlin connections.

The bolted connections between purlins and rafters are designed using allowable stress design. With the use of the higher coefficients for the edge strips and corners for the wind loads, it is reasonable to assume that correctly installed purlin-rafter connections will have the capacity to support additional loads of the three adjacent purlins before they yield.

To evaluate the effects of purlin anchorage failures, it is assumed in this study that four consecutive purlins become detached from the rafter in a wind storm. The loads of three purlins are transferred to one adjacent purlin on one side and the load of the fourth purlin is transferred to the adjacent purlin on the other side. The uneven distribution is justified by the variation in the peak pressures. This uneven redistribution of wind loads may stay on the frame a short time and may lead to additional purlin anchorage failures. However, it is felt that even for the short duration, a well designed frame should be capable of resisting this assumed distribution of wind loads without being excessively overstressed.

Since the wind continues to exert the same uplift, the loads for purlins whether inactive or intact conditions are kept the same as per MBMA 1986. The frames are then analyzed using this redistributed load. The changes in the moment profile and the unity check are plotted. Findings are given below.

4.3.1 Low-Slope Building

The effect of inaction of four purlins is examined for the low-slope building. The frame is designed for the partially enclosed condition (Figure 3.7). The inclined length of the rafters is 57.03 feet. A total of 14 purlins are attached to the top flange of the rafter at a spacing of 4.5 feet on centers. Purlin anchorage failures are assumed for four consecutive purlins located at 32.7', 37.2', 41.7', and 46.2'. The loads are redistributed as explained above. The rafter moments and the unity check for the uniform and redistributed load cases are shown in Figures 4.14 and 4.15. At a section 46 feet from the knee, the moment changed from -234

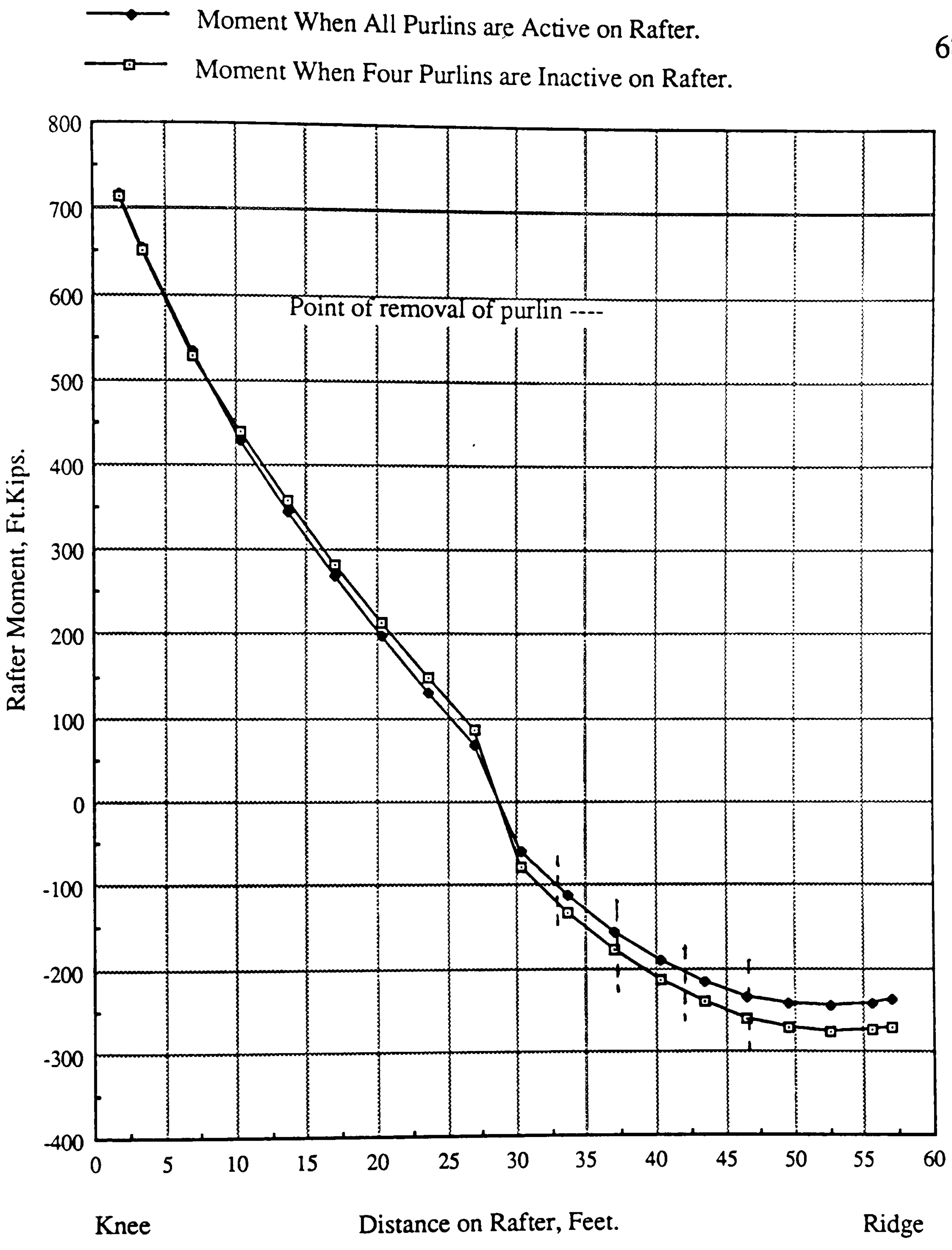


Figure 4.14 Rafter Moments for Low-Slope Frame. (Span 120 ft., Wind Speed=100 mph., Roof Slope .5/12, Int. Zone, Partially Enclosed)

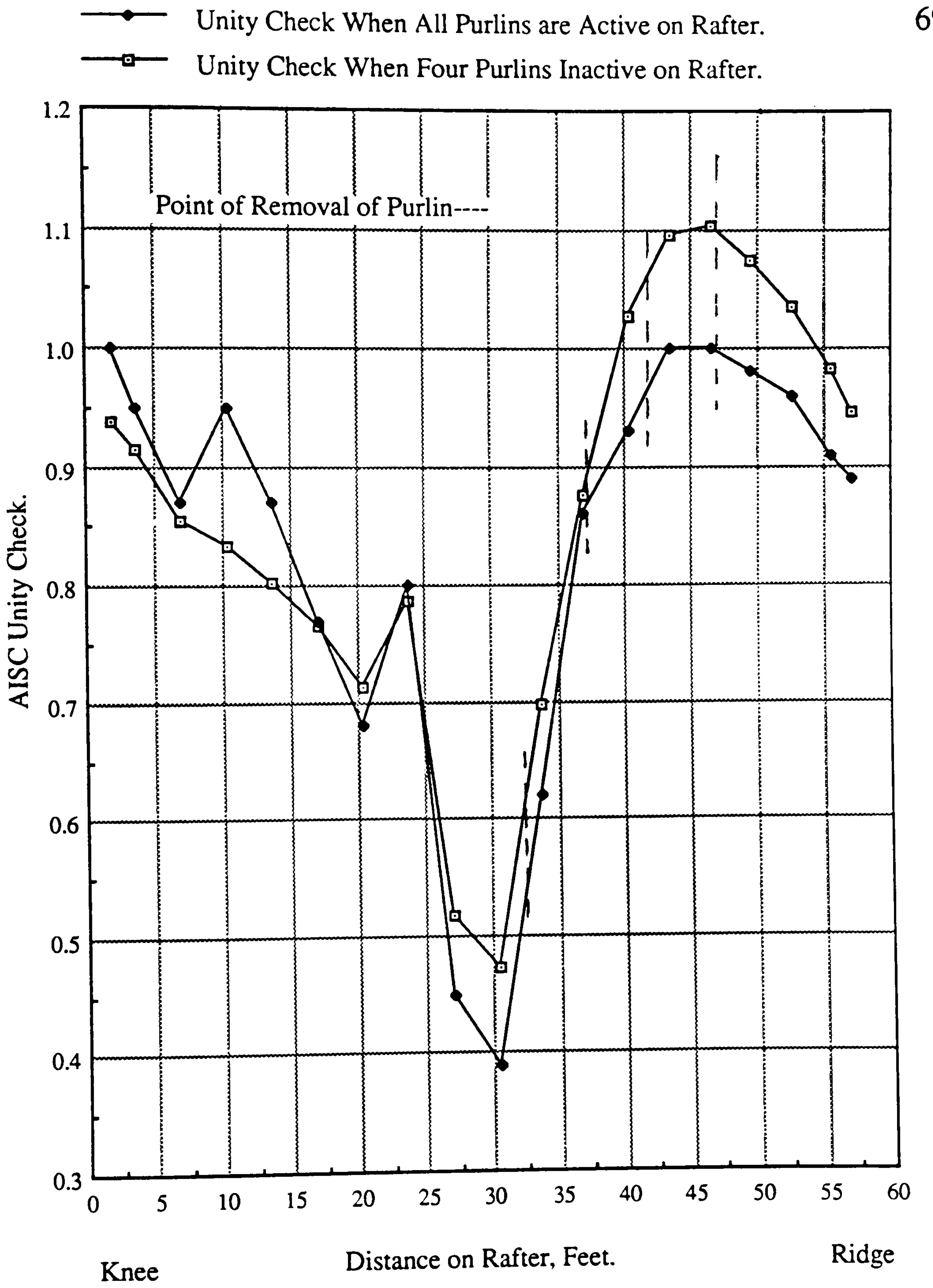


Figure 4.15 Unity Check for Low-Slope Frame.
(Span 120 ft., Wind Speed=100 mph, Roof Slope=.5/12, Int. Zone, Partially Enclosed)

to -261 ft-kips. At this section the unity check increased from 1.00 to 1.103. Substantial increases in stresses are observed at many other sections also (Figure 4.15). The effect of the load redistribution on the rafter had a negligible effect on the moments for the column. No increase in design vertical reaction or horizontal thrust at the base connection is observed.

4.3.2 High-Slope Building

The effect of inaction of four purlins is examined. The frame is designed for the partially enclosed condition (Figure 3.10). The inclined length of the rafters is 67.04 feet. Fourteen purlins are attached to the top flange of the left rafter at a spacing of 5.21 feet on center. It is assumed that purlin anchorage failures occur for purlins located at 17.9', 23.1', 28.3', and 33.5'. Wind loads are redistributed as explained above.

At a section 26' from the knee bending moment increased from -445 to -507 ft-kips. At this section the unity check increased from 0.86 to 0.97. Substantial increases in the moments and stresses are observed at many other sections also (Figures 4.16 and 4.17). The effect of load redistribution on the rafter had negligible effect on columns. No increase in design vertical or horizontal reaction at the base connections is observed.

4.4 Unexpected Overload Due to High Winds

Overload on a metal building system occurs when it is subjected to higher wind speeds than those specified by the code. When an owner elects to construct a metal building to minimum standards, he should be aware that the building could be overloaded by high winds one or more times

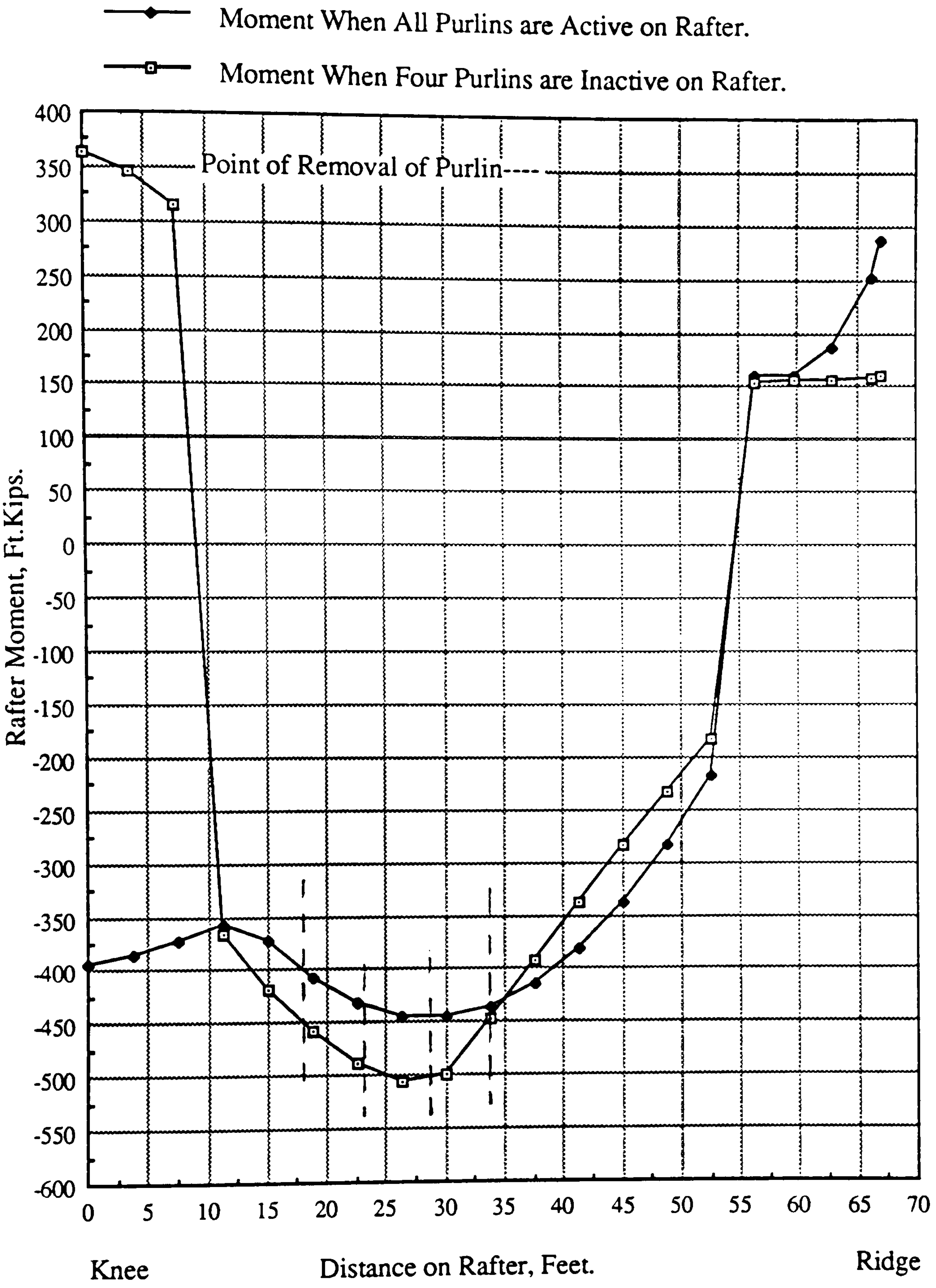


Figure 4.16 Rafter Moments for High-Slope Frame. (Span 120 ft., Wind Speed=100 mph, Roof Slope 7/12, Int. Zone, Partially Enclosed)

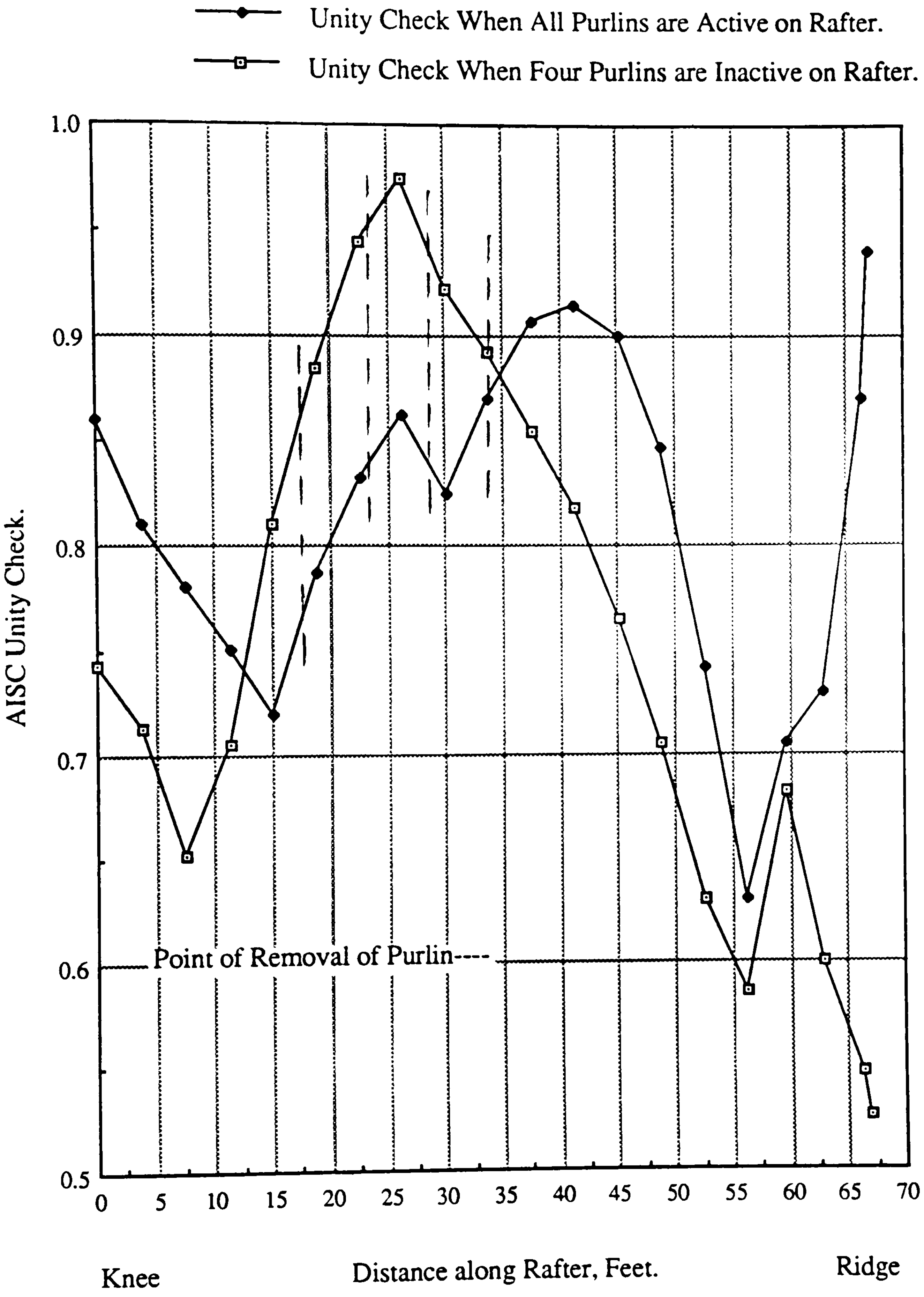


Figure 4.17 Unity Check for High-Slope Frame. (Span 120 feet, Wind Speed=100 mph, Roof Slope 7/12, Int. Zone, Partially Enclosed)

during the life of the building. Hurricane winds are often more than the minimum design wind speeds set by MBMA. For example, the design wind speed for Corpus Christi, Texas is 95 mph, whereas, during the period 1916-1970 Corpus Christi was hit by three devastating hurricanes with fastest-mile winds up to 120 mph (Simiu and Scanlan 1978). The owner should be aware of the consequences of the overloading and weigh the possibility of damage to his facility against the additional costs of higher design loads.

To help the owners of metal buildings, it is, therefore, considered logical in this study to evaluate the effects of high wind speeds. Since at high wind speeds the building may lose wall or roof panels, frames designed for the partially enclosed condition for 100 mph wind loads (Figures 3.7 and 3.10) are considered in this analysis. Wind loads for 110, 120, 130 and 140 mph winds are applied to the frames designed for 100 mph wind loads of a partially enclosed building. Increases in moments and the unity checks at many points along column and rafter are computed. The weakest points where moments or the unity check increased considerably are identified. Increases in the vertical and horizontal reactions at base are also computed. Furthermore, frames are designed for wind speeds of 110, 120, 130 and 140 mph for both roof slopes. The weights for the higher wind resistant frames are computed. Findings are given below.

4.4.1 Low-Slope Building

Figure 4.18 shows the increases in the unity check along the rafter with an increase in wind speed from 100 mph to 140 mph. A careful study of

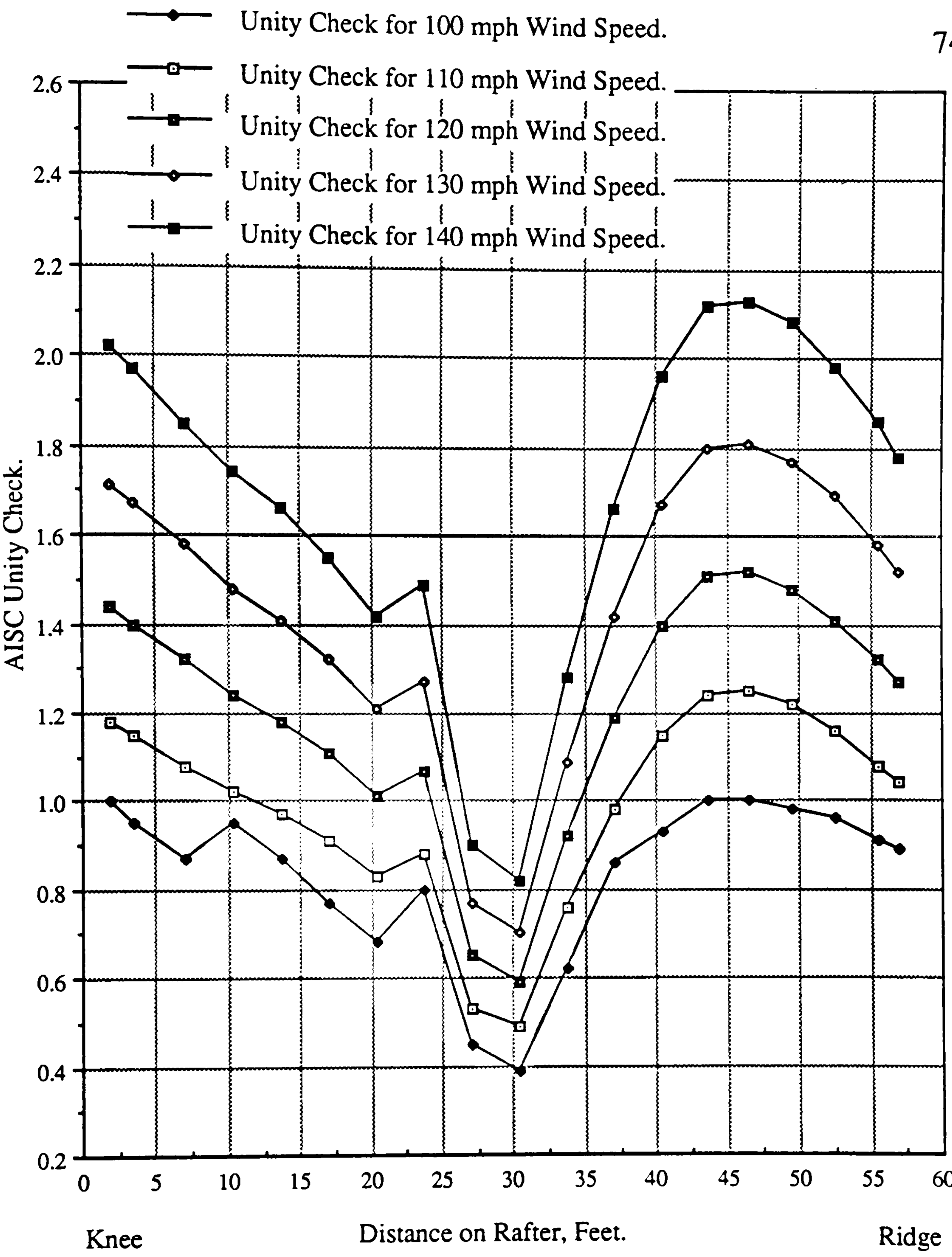


Figure 4.18 Unity Check for Low-Slope Frame.
(Span 120 ft., Design Wind Speed=100 mph,
Roof Slope .5/12, Int. Zone, Partially Enclosed)

Figure 4.18 reveals that with only a 10 mph increase in the design wind speed, a 32 feet length of the rafter becomes unacceptable as per the allowable stress design criteria. This unacceptable length increases to 52 feet as the wind speed increases to 140 mph. It is also noted that the rates of increases are not uniform, e.g., when the design wind speed increases from 100 to 140 mph, at a 10.3 feet distance an 83%, and at 46.5 feet a 113% increase in the unity check is noted. The variation in the increase in the unity check is due to the fact that at 10.3 feet DL + WLR loading condition gave the highest increase, whereas at 46.5 feet distance DL + WLL loading condition gave the maximum increase for 140 mph winds.

The effect of a higher wind speed on the design of base connection is found to be substantial. The vertical reaction on base connection increased from -44.0 to -94.6 kips for the wind speed increase from 100 to 140 mph. The horizontal reaction on base connection increased from 50.6 to 108.3 kips. The DL + WLL case gave the maximum horizontal and vertical reactions. It may be noted that vertical and horizontal reactions increased by a factor of 2.14, whereas the velocity pressure increases only by a factor of 1.96. This is because the dead loads are the same for 100 and 140 mph cases. Therefore the ratio of dead load + wind load for 140 mph winds to that of dead load + wind load for 100 mph winds works out to be more than the ratio of velocity pressures.

4.4.2 High-Slope Building

Figure 4.19 shows the increases in the unity check along the rafter with the increase in wind speed from 100 to 140 mph. A careful study of Figure 4.19 reveals that with only ten mph increase in the wind speed,

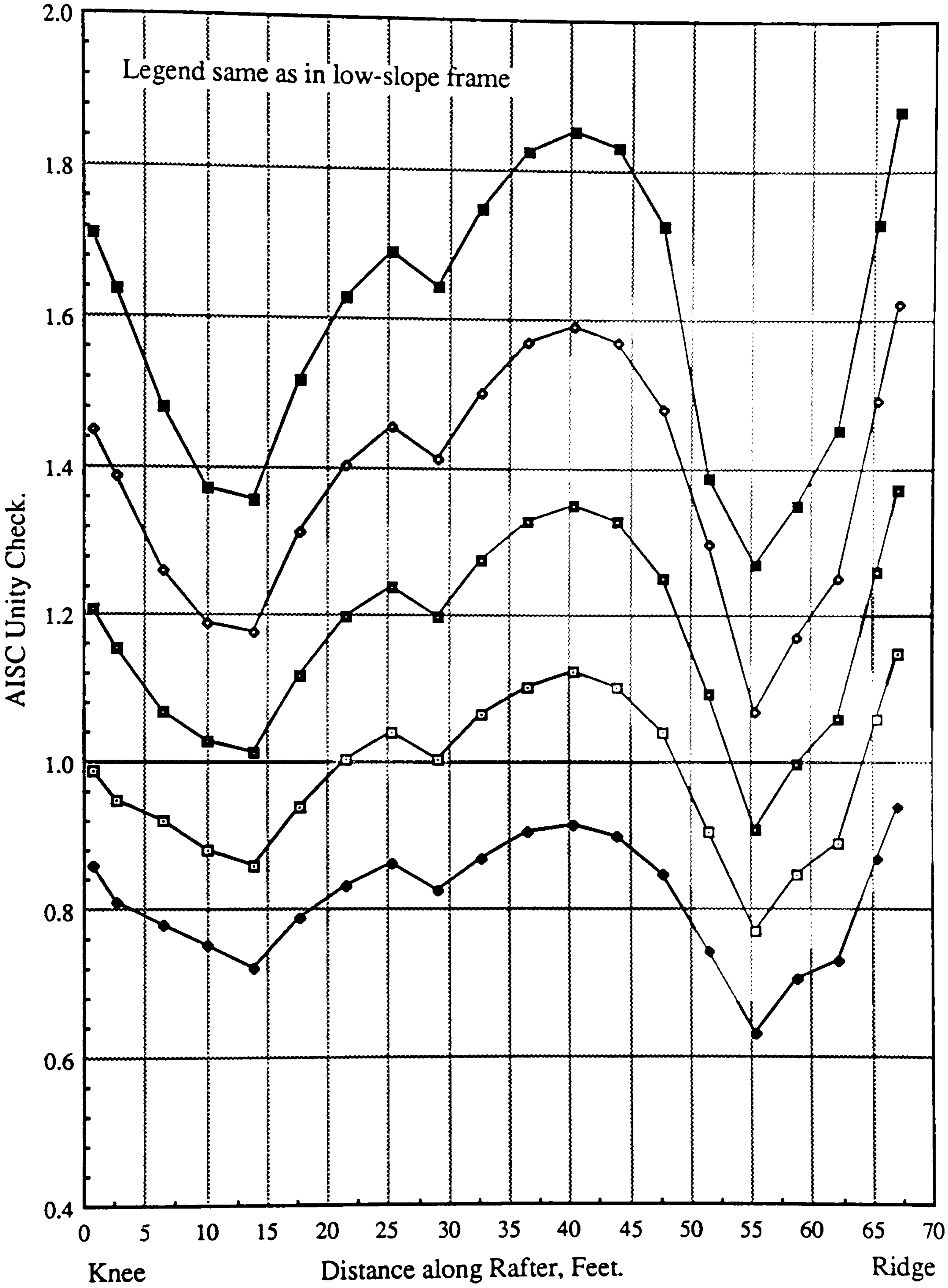


Figure 4.19 Unity Check for High-Slope Frame.
(Span120 ft., Design Wind Speed=100 mph.,
Roof Slope=7/12, Int. Zone, Partially Enclosed)

middle 25 feet of the rafter becomes unacceptable as per the allowable stress design criteria. All the points on the rafter exceed the unity check at a wind speed of 130 mph. The increases in the unity check are found to be not proportional to the increase in the wind speed. For example when wind speed increases from 100 to 140 mph, at 10.1 feet 82%, and at 47.7 feet 103% increase in the unity check is noted. Also, the unity check is maximum for DL + WL2 loading case at 10.1 feet and for DL + WR2 wind loading case at 47.7 feet for the 140 mph winds.

Effect of higher wind speed on the design of base connection is found to be substantial. The design vertical reaction on one base connection increased from 49.5 to 106.6 kips for the wind speed increase from 100 to 140 mph. The DL + WLL case gave the maximum vertical reactions for 100 and 140 mph winds. The horizontal reaction on one connection increased from 23.6 kips for DL + WL3 case to 48.4 kips for DL+WL2 case. It may be noted that because of three different wind load cases and the constant dead load the maximum vertical and horizontal reactions will not increase directly in proportion to the increase in the velocity pressure. Further, the ratios of increases in the horizontal and vertical reactions for 140 and 100 mph winds is found to be higher than the ratios of their velocity pressures.

4.4.3 Design of Frames for High-Wind Speeds

Frames with both roof slopes are designed for wind speeds of 110, 120, 130 and 140 mph. Weights of the frames designed for various wind speeds are shown in Figure 4.20. Although the wind loads and design moments can be assumed to increase with the second power of wind speed, the

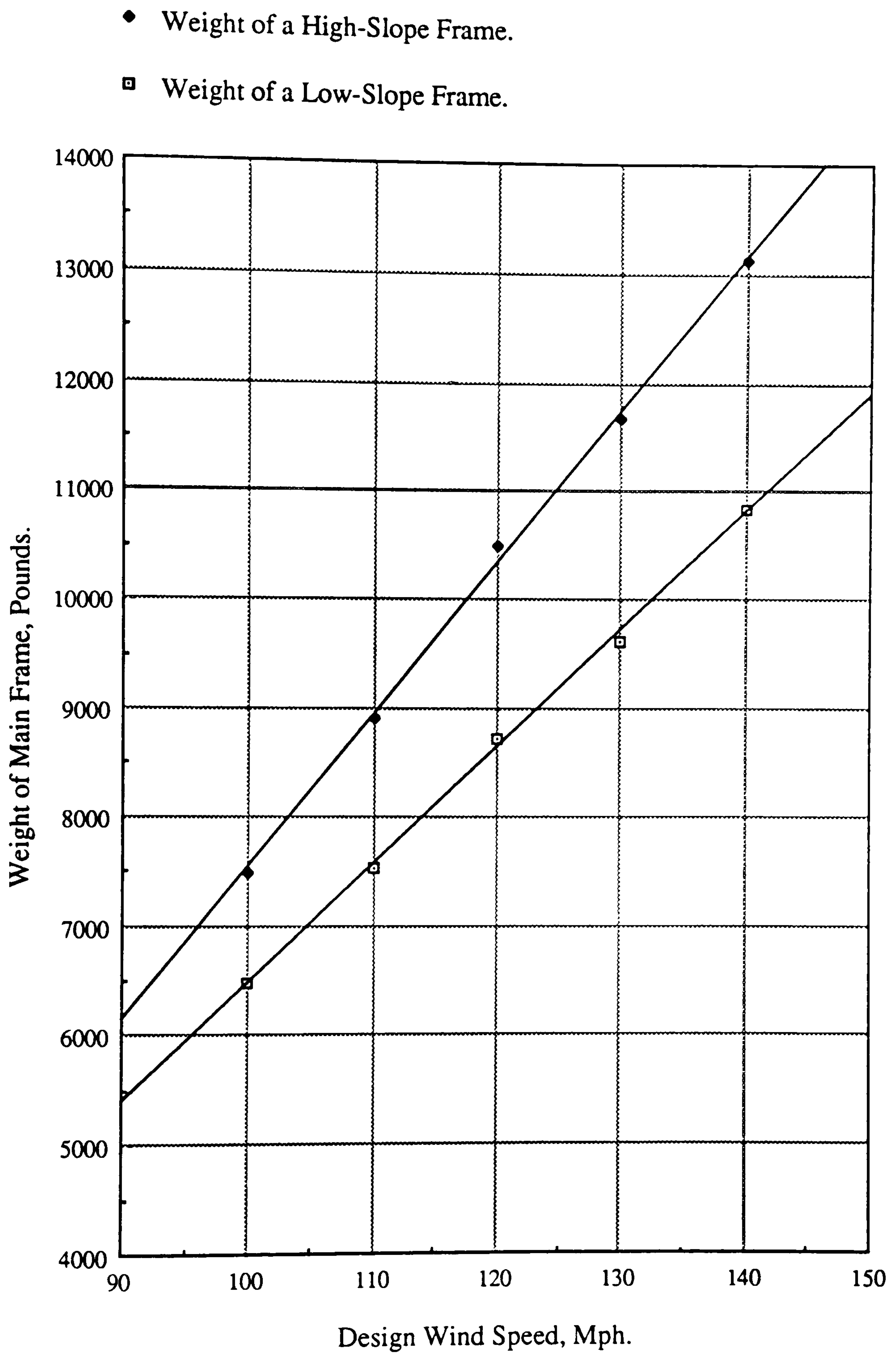


Figure 4.20 Weight of Main Frame Versus Design Wind Speed. (Span 120 Feet, Int. Zone, Partially Enclosed)

weights of frames increase approximately linearly with the wind speed. This is probably due to the fact that the section modulus and the weight of the built-up frames do not increase in the same ratios. The following linear equations describe the relationships between weight and wind speed of the frames.

$W = -4335.4 + 108.0 \times V$ (Low-Slope Frame).

$W = -6489.0 + 140.2 \times V$ (High-Slope Frame).

In the above equations, W, represents the frame weight in pounds, and V, represents the design wind speed in mph. It can be seen that the slope of the line is steeper for the high-slope frame. A cost increase of $ 0.78 per square foot for the low-slope frame and $1.56 per square foot for the high-slope frame is estimated for increase in the design wind speed from 100 to 140 mph. These cost increases are only for the structural frames. Stronger purlins, girts and cladding needed to resist the higher wind speeds may result in additional costs. It may be noted that purlins, girts and cladding do not cost more than one-third of the overall cost of the building (Perry 1989). Therefore the increase in the cost for purlins and cladding will be much less than that for the structural frames.

4.5 Effect of Tapered Members

Main wind-force resisting system pressure coefficients (MBMA 1986) are based on the extensive wind tunnel studies performed at the University of Western Ontario (UWO). The coefficients were obtained using influence lines for frames having prismatic members. In reality the frames are tapered to reduce the weight of steel. Therefore, the coefficients based on influence lines of prismatic frame may lead to unrealistic pressure coefficients. In this analysis an attempt is made to check whether the assumption of prismatic members leads to conservative and safe coefficients for the design purposes.

Researchers at the University of Western Ontario considered both two- and three-hinge frames having prismatic members. In order to draw an influence line for a frame, the relative moment of inertias of columns and rafters are needed. Researchers at the University of Western Ontario assumed moments of inertia of columns and rafters in ratios of their respective lengths. The detailed procedure for obtaining these coefficients is explained in section 3.2.

However, it may be noted that there is a substantial difference in the frames assumed and the frames being constructed (Figures 1.1, 3.7, 3.10). To evaluate the adequacy of the assumption of a prismatic frame a comparison of design parameters of tapered members and prismatic members is made. The design parameters, e.g., moments at knee and ridge, vertical and horizontal reactions at a base connection are computed for prismatic as well as tapered frames. The two-hinge and three-hinge prismatic frames are considered as in the UWO study. The moments of inertia for column and rafter are assumed as I and 6I for 20 feet high 120 feet wide frame for the low-slope and I and 6.95I for the high-slope frame. Design parameters are then computed for two hinge and three hinge prismatic frames.

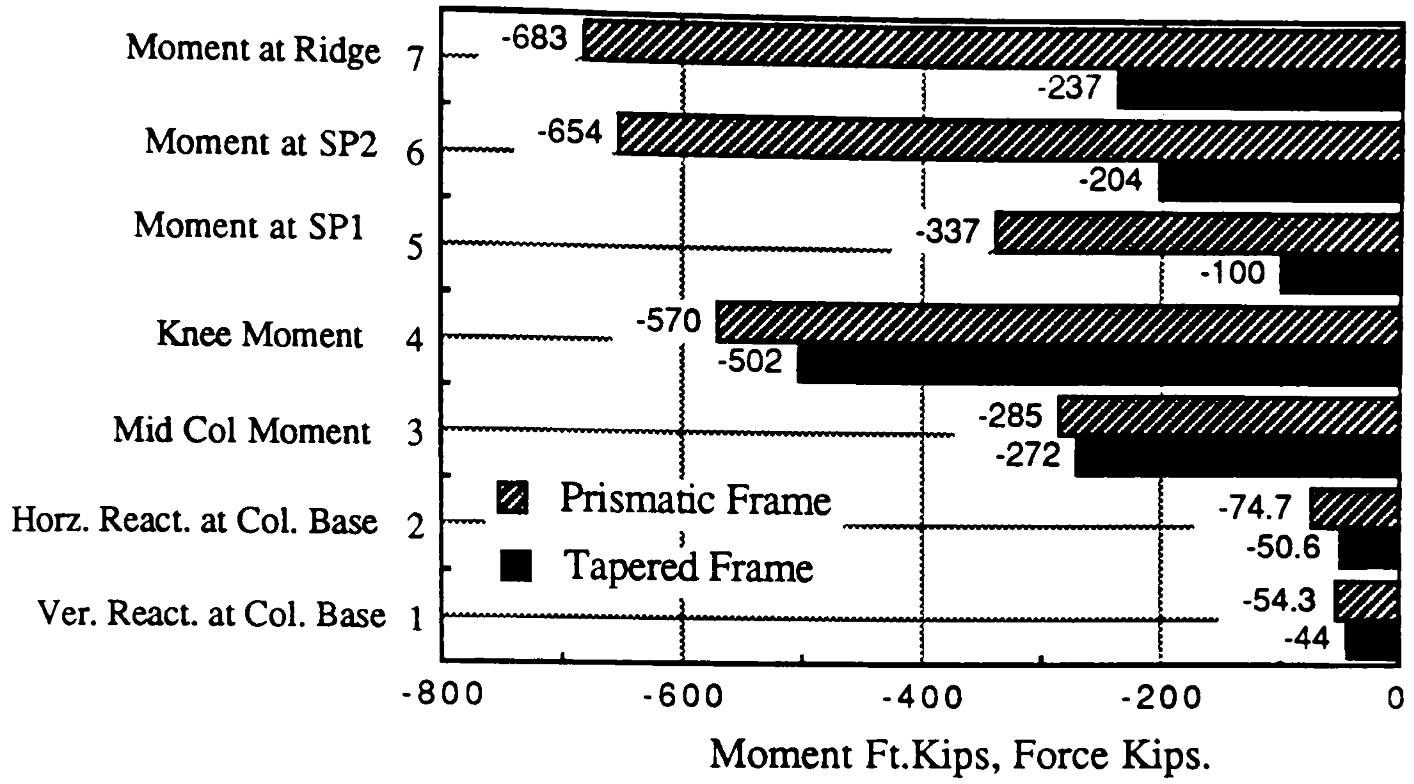
Comparison is made by computing the same design parameters for the two-hinge tapered frames used in this study. Since the objective is to make a comparison of theoretical and realistic frames the self weight of the tapered frames is included in the analysis. Findings are given below.

4.5.1 Low-Slope Building

Figure 4.21 shows the comparison of prismatic frames with tapered frame used in this study. The shaded bars indicate the range for the computed design parameters for two-hinge and three-hinge prismatic frames. All the four wind loading cases and the DL+LL case are applied to the prismatic frames. The shaded bars therefore indicate the ranges for the design parameters for 10 loading conditions (5 for two hinge and 5 for three hinge). The dark bars indicate the range for the computed design parameters for the tapered frame used in this study. Since the ranges computed for tapered frame are much less than the ranges computed for the prismatic frames, for all the design parameters, it can be concluded that the assumption of using prismatic members is safe and conservative for low-slope frames.

4.5.2 High-Slope Building

Figure 4.22 shows the comparison of design parameters for prismatic frame with tapered frame used in this study. The range for the tapered frame for moments produced at ridge exceeded the range for the prismatic members by 28% for positive and 8% for negative values. Range for the knee moment for tapered frame exceeded than that of the prismatic frame by 55% for the negative moments (Figure 4.22). At the middle of the column, the range for the tapered frame exceeded that of the prismatic frame by 28% for negative moments. It appears that the assumption of prismatic members may lead to pressure coefficients that under-design the ridge for moments and knee and columns for inner flange bracing. Since the computed range for the tapered frame exceeded that of the prismatic



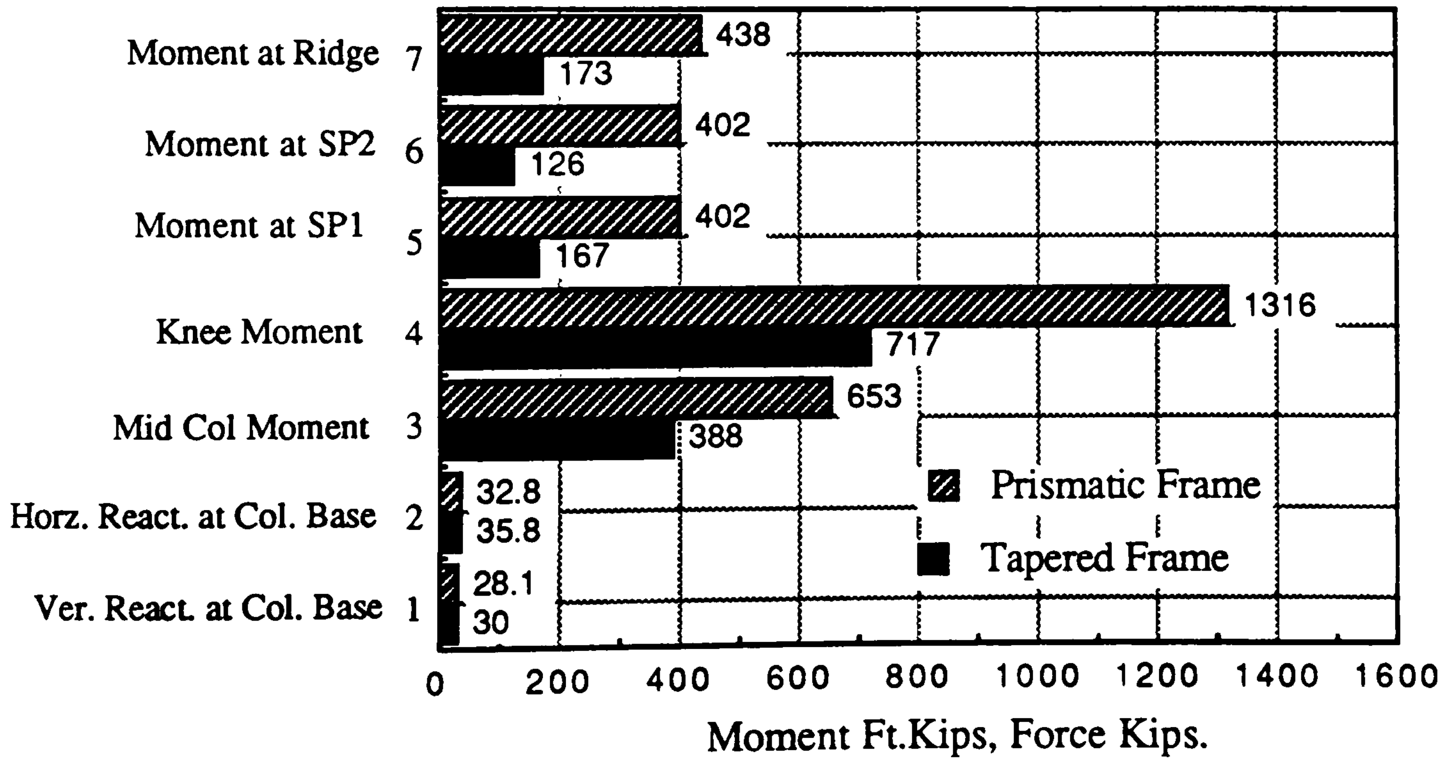
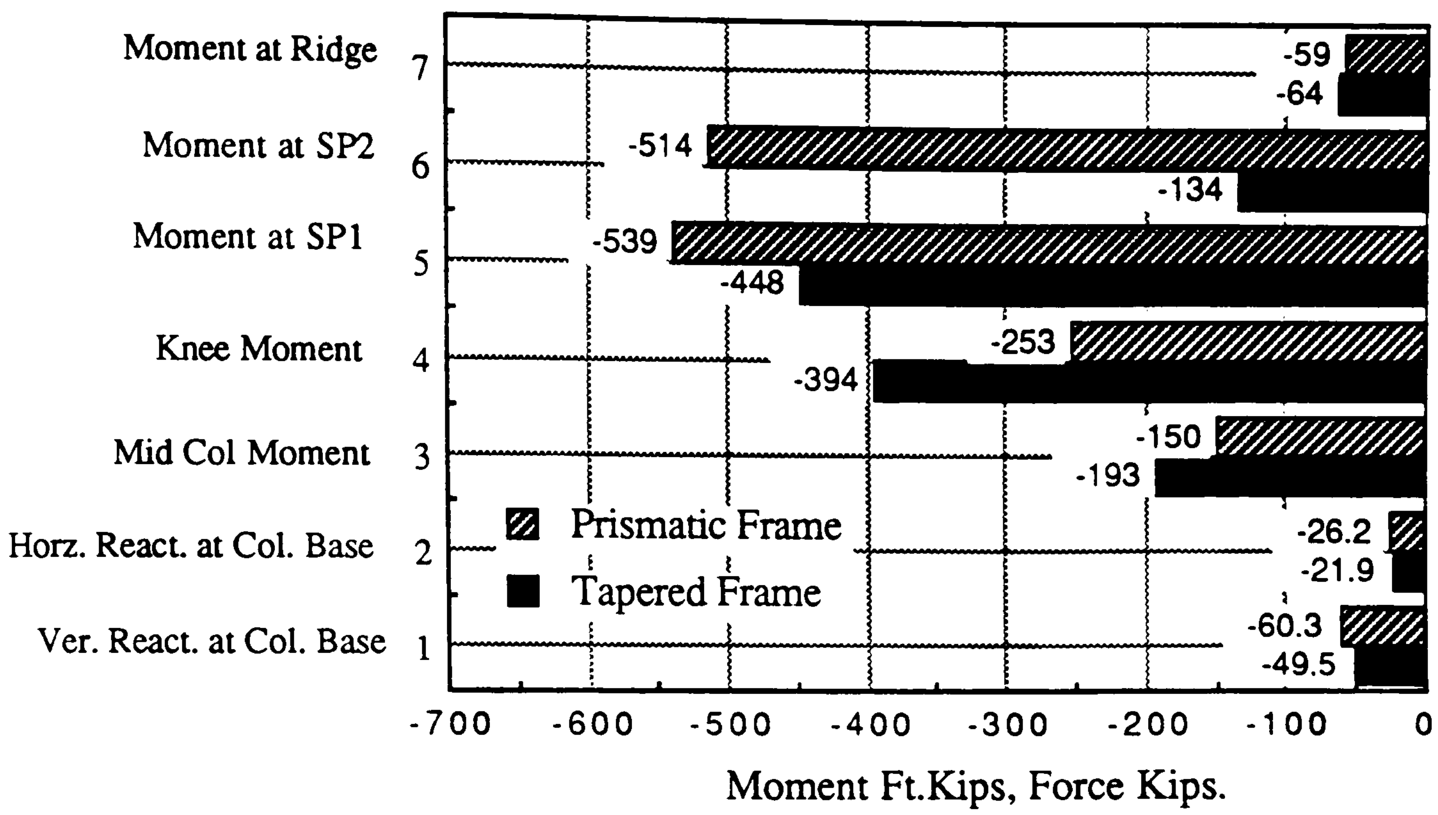


Figure 4.21 Comparison of Low-Slope Prismatic and Tapered Frames. (Span 120 ft, Wind Speed=100 mph, Roof Slope .5/12, Int. Zone, Partially Enclosed)



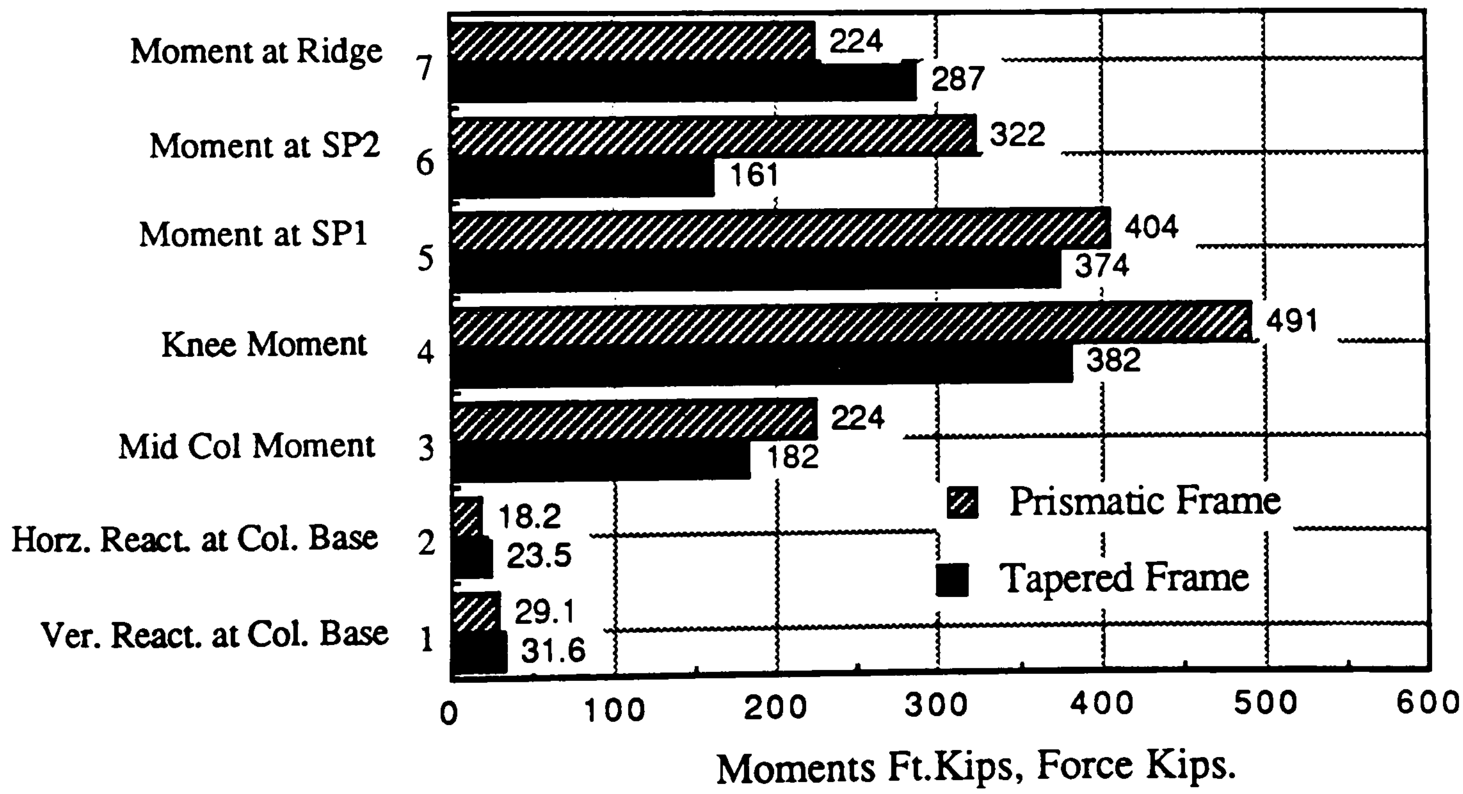


Figure 4.22 Comparison of High-Slope Prismatic and Tapered Frames.
(Span 120 ft, Wind Speed=100 mph, Roof Slope 7/12, Int. Zone, Partially Enclosed)

frame, it is obvious that assumption of two- and three-hinge prismatic frames provides unconservative envelopes for some design parameters.

CHAPTER V
SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary

The causes of damage to metal building systems in extreme windstorms have been known for quite some time. At the present time, the literature available on metal building systems is devoid of studies that show the changes in the various design parameters for buildings that become deficient in wind storms. Such studies can help designers to design better wind resistant buildings and code writing bodies to specify better design criteria. Of course, for this to happen, studies indicating quantitatively the changes in the design parameters and the related cost increases will be needed.

This research examines several types of metal building wind damage and reaches at quantitative conclusions. The following problems are addressed in this study:

1. Breach of the building envelope.
2. Bracing omitted from compression flanges.
3. Purlin anchorage failures.
4. Unexpected overload due to high winds.
5. Effect of tapered members.

To study the effects of above mentioned problems a low-slope frame having a roof slope of .5/12 and a high-slope frame having a roof slope of 7/12 are considered in this study. The frames are designed for enclosed and partially enclosed conditions, interior zone using 100 mph design wind speed, for two roof slopes. A 120'-span and 30'-bay spacing is used. Latest design criteria according to MBMA 1986 is used. Frames are optimized using computer programs presently used by the metal building industry. Commercially available plate sizes are specified. Frames represent typical designs that would have been produced by the metal building industry.

Effects of the above mentioned problems are then modelled and studied by making changes in the loads or structural configuration. Changes in the design moment profile and AISC unity check are computed along columns and rafters. Results indicate that the frames designed according to the minimum design criteria of MBMA 1986 are not adequate for resisting the above changes. The study also shows that metal building frames can be economically designed to resist wind loads on enclosed and partially enclosed buildings.

5.2 Conclusions

Findings of this study suggest the following conclusions:

1. Metal buildings should be designed as partially enclosed.

Breach in the building envelope is the most damaging effect of a windstorm and adversely affects the main frames in many ways. Breach of the building envelope occurs due to failure of personnel or overhead doors, or due to the impact of storm generated missiles. It is customary to design metal buildings as enclosed. A breach will increase and reverse the sign of bending stresses in the column as well as in the rafter for the low-slope frame. It will induce yielding at the ridge of high-slope frames. A breach will also increase the vertical and horizontal reactions up to 60% and 40% at one column base connection, increases that are sufficient to initiate failure of the base connection. Cost increases of partially enclosed buildings as compared to enclosed buildings are small.

2. Omission of flange bracing initiates failure by local buckling because stresses exceed equivalent allowable stresses by a factor of two or more.

Flange braces are sometimes left out or installed incorrectly by the steel erectors. This reduces the capacity of the rigid frame particularly under wind loads. The reduction in capacity was modelled by introduction of smaller compression flange. Computations indicated that a lack of lateral restraint of the compression flange at two adjacent purlin points initiates local buckling for the low-slope as well as the high-slope roof. Further, the computations indicated an increase in the unity check of 8% (up to a 3 ksi increase in bending stress) for the low-slope roof and of 28% (up to an 11 ksi increase in bending stress) for the high-slope roof at locations farther away from the laterally unbraced section.

3. Purlin anchorage failures, leading to a condition where wind loads are not transferred uniformly to the rafter, increases bending stresses significantly on the rafter.

Purlins are usually anchored to rafters using A307 low-carbon steel bolts. A very common condition in hurricanes and tornadoes is the purlin anchorage failures. Although, the wind continues to exert the same uplift, loads are not transferred uniformly to the main frame. It is found in this study that redistribution of wind uplift may increase the unity check by 10%. That indicates an increase in bending stresses up to 4 ksi at some points. Changes in the vertical and horizontal reaction at the base connection and the stresses in the column are found to be negligible.

4. Overloading the frame due to wind speeds higher than those specified by the code will initiate yielding at several points on the frame.

Metal buildings are designed to the minimum standards as specified by MBMA. The owner should be aware that the buildings are likely to be overloaded sometimes during their life. He should also be aware of the consequences of designing for the minimum standards. Computations indicated that a 10 mph increase in the wind speed for the low-slope roof and a 20 mph increase for the high-slope roof brought working stresses greater than or equal to yield stress at several points. Frames are indeterminate to a single degree and yielding at some points will form a mechanism and subsequent collapse of the frame. The horizontal and vertical reactions on a base connection increase more than the increase in the velocity pressure.

Computations based on an initial cost of $12 per square foot indicated that cost increases will be $0.78 per square foot for the low-slope and $1.56 per square foot for the high-slope roof frames when designing for 140 mph wind. These cost increases are based on the main frame only. Designing for higher wind speeds, therefore, will increase construction costs substantially.

5. The assumption of using two-hinge and three-hinge frames having prismatic members for the derivation of the pressure coefficients by UWO is found to be safe except for the high-slope roof frame used in this study.

5.3 Recommendations for Further Study

Following are the recommendations for further research:

1. Except for the breach of the building envelope the other problems associated with the high winds are examined for the designs for the partially enclosed buildings. It will be worthwhile to examine the effects of these problems for enclosed buildings.

2. This study does not address the design for parts and portions and cladding. The effect of these problems i.e. overstress and cost increases for parts and portions and cladding should be addressed.

3. Other problems associated with the high-winds e.g. strut purlin failures, failure of column anchor bolts or cross bracing should be addressed.

4. Effects of a higher degree of redundancy should be examined.

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GLOSSARY

Base Plate--A plate attached to the bottom of a column which rests on a foundation or other support, usually secured by anchor bolts.

Bay--The space between the main frames measured normal to the frame.

Bearing Plate--A steel plate that is set on the top of a masonry support on which a beam or purlin can rest.

"C" Section--A member formed from steel sheet in the shape of a block "C", that may be used either singularly or back to back.

Cold Forming--The process of using press brakes or rolling mills to shape steel into desired cross sections at room temperature.

Covering--The exterior metal roof and wall paneling of a Metal Building System.

Dead Load--The weight of the Building System construction consisting of members such as framing and covering, plus all collateral loads.

Design Loads--The loads expressly specified in the contract documents which the Metal Building System is designed to safely resist.

Design Professional--The Architect or Engineer responsible for the design of a Construction Project.

Eave--The line along the sidewall formed by the intersection of the planes of the roof and wall.

Eave Height--The vertical dimension from finished floor to the eave.

Elastic Design--A design concept utilizing the proportional behavior of materials when all stresses are limited to specified allowable values in the elastic range.

End Bay--The bays adjacent to the endwalls of a building. Usually the distance from the endwall to the first interior main frame measured normal to the endwall.

End Zone--The surface area of a building along the roof at the endwall and at the endwall corners where the wind loads on main frames are greater than at other areas.

Engineer/Architect of Record--The engineer or architect who is responsible for overall design of the building project.

Gable Roof--A roof consisting of two sloping sides that form a ridge and a gable at each end.

Important Factor--A factor that accounts for the degree of hazard to human life and damage to property.

Inner Flange--Flange of the built up section of columns and rafters towards the interior of the building. Flange bracings are attached to inner flanges of the frames.

Insulation--Any material used in building construction to reduce heat transfer.

Knee--The connecting area of a column and rafter of a structural frame such as a rigid frame.

Longitudinal--The direction parallel to the ridge or sidewall.

Low Rise Building--A description of a class of buildings usually less than 60' eave height. Commonly, they are single story, but do not exceed 4 stories.

Main Framing--The main load carrying members of a structural system.

Manufacturer--A party who designs and fabricates a Metal Building System.

Metal Building System--A complete integrated set of mutually dependent components and assemblies that form a building including primary and secondary framing, covering and accessories, and are manufactured to permit inspection on site prior to assembly or erection.

Overhead Doors--Doors constructed in horizontally hinged section. They are equipped with springs, tracks, counter balancers, and other hardware which roll the sections into an overhead position, clear of the opening.

Parts and Portions--Members that transmit loads to the main frames. They include girts, joists, purlins, studs, covering, end wall columns and end wall rafters of bearing end frames, masonry walls when acting as other than shear walls, coverings, and fasteners.

Peak--The uppermost point of a gable.

Pinned Base--A column base that is designed to direct the flow of water out through the face of the gutter rather than through a downspout.

Pin Connection--A connection designed to transfer axial and shear forces between connecting members, but not moments.

Plastic Design--A design concept based on multiplying the actual loads by a suitable load factor, and using the yield stress as the maximum stress in any member, and taking into consideration moment redistribution.

Positive Moment--Bending Moment that creates compression in the outer flanges of columns and Rafters.

Purlin--A horizontal structural member which supports roof covering.

Rafter--The main beam supporting the roof system.

Rigid Frame--A structural frame consisting of members joined together with moment connections so as to render the frame stable with respect to the design loads, without the need for bracing in its plane.

Roll-up Door--A door that opens by traveling vertically.

Roof Covering--The exposed exterior roof surface consisting of panels.

Roof Pitch--Ratio of rise to building width for gable roofs.

Roof Slope--The angle that a roof surface makes with the horizontal. Usually expressed in units of vertical rise to 12 units of horizontal run.

Roof Snow Load--That load induced by the weight of snow on the roof of the structure.

Section Modulus--A geometric property of a structural member. It is used in design to describe the flexural strength of a member.

Self Tapping Screw--A fastener which taps its own threads in a pre-drilled hole.

Single Span--A building or structural member without intermediate support.

Stiffener--A member used to strengthen a plate against lateral or local buckling. Usually a flat bar welded perpendicular to the longitudinal axis of the member.

Tapered Members--A built up plate member consisting of flanges welded to a variable depth web.

Tensile Strength--The longitudinal pulling stress a material can bear without tearing apart.

Tributary Area--The area which contributes load to a specific structural component.

Uplift--Wind load on a building which causes a load in the upward direction.

Width--The dimension of the building measured parallel to the main framing from sidewall to sidewall.

"Z" Section--A member cold formed from steel sheet in the shape of a block "Z."

APPENDIX A
DESIGN WIND AND SNOW LOADS IN HURRICANE PRONE REGIONS

DESIGN WIND AND SNOW LOADS IN HURRICANE PRONE REGIONS[1]				
SNOW LOAD LBS/SQ FT	WIND LOAD MILES/HOUR	MILES FROM COAST LINE[2]	EARTH QUAKE ZONE	COUNTY NAME
ALABAMA				
0	94	12	0	BALDWIN
0	93	15	0	MOBILE
FLORIDA				
0	109	7	0	BROWARD
0	100	4	1	CHARLOTTE
0	104	4	0	COLLIER
0	101	7	0	FRANKLIN
0	101	5	0	GULF
0	100	46	0	HENDRY
0	101	4	0	LEE
0	101	2	1	MANATEE
0	100	10	0	MARTIN
0	113	8	0	PALM BEACH
0	101	5	1	PINELLAS
0	101	1	1	SARASOTA
GEORGIA				
0	91	12	1	CAMDEN
0	91	16	2	CHATHAM
0	91	17	1	GLYNN
0	92	16	1	MCINTOSH

(1) Low-Rise Building Systems Manual (MBMA 1986).

(2) Distance is measured from coastline to the population center of the county.

DESIGN WIND AND SNOW LOADS IN HURRICANE PRONE REGIONS				
SNOW LOAD LBS/SQ FT	WIND LOAD MILES/HOUR	MILES FROM COAST LINE	EARTH QUAKE ZONE	COUNTY NAME
HAWAII				
0	110	-	0	HONOLULU
0	110	-	0	KAUAI
LOUISANA				
0	96	32	1	ASCENSION
0	99	32	1	ASSUMPTION
0	97	4	1	CAMERON
0	98	9	1	IBERIA
0	96	39	1	IBERVILLE
0	99	3	1	JEFFERSON
0	95	28	1	LAFAYETTE
0	103	19	1	LAFOURCHE
0	95	24	1	LIVINGSTON
0	99	6	1	ORLEANS
0	103	16	1	PLAQUEMINE
0	102	10	1	ST. BERNARD
0	101	7	1	ST. CHARLES
0	98	26	1	ST. JAMES
0	98	11	1	ST. JOHN THE BAPTIST
0	95	30	1	ST. MARTIN
0	101	10	1	ST. MARY
0	95	13	1	ST. TAMMANY
0	103	25	1	TERREBONNE
0	96	18	1	VERMILLION
0	91	34	1	WASHINGTON
0	93	57	1	WEST BATON ROUGE

DESIGN WIND AND SNOW LOADS IN HURRICANE PRONE REGIONS				
SNOW LOAD LBS/SQ FT	WIND LOAD MILES/HOUR	MILES FROM COAST LINE	EARTH QUAKE ZONE	COUNTY NAME
NORTH CAROLINA				
10	97	14	1	BRUNSWICK
10	104	6	1	CARTERET
10	95	12	1	CRAVEN
10	100	8	1	DARE
10	99	5	1	HYDE
10	96	17	1	ONSLOW
10	99	9	1	PAMLICO
10	96	3	1	TYRRELL
SOUTH CAROLINA				
0	94	11	3	BEAUFORT
5	95	21	3	BERKELEY
0	98	8	3	CHARLESTON
5	96	15	3	GEORGETOWN
10	95	8	2	HORRY
TEXAS				
0	97	9	0	BRAZORIA
0	92	4	0	CALHOUN
0	97	2	0	GALVESTON
0	91	22	0	JEFFERSON
0	92	19	0	MATAGORDA
0	91	29	0	ORANGE

APPENDIX B
VALIDATION OF THE "DESIGN" COMPUTER PROGRAM

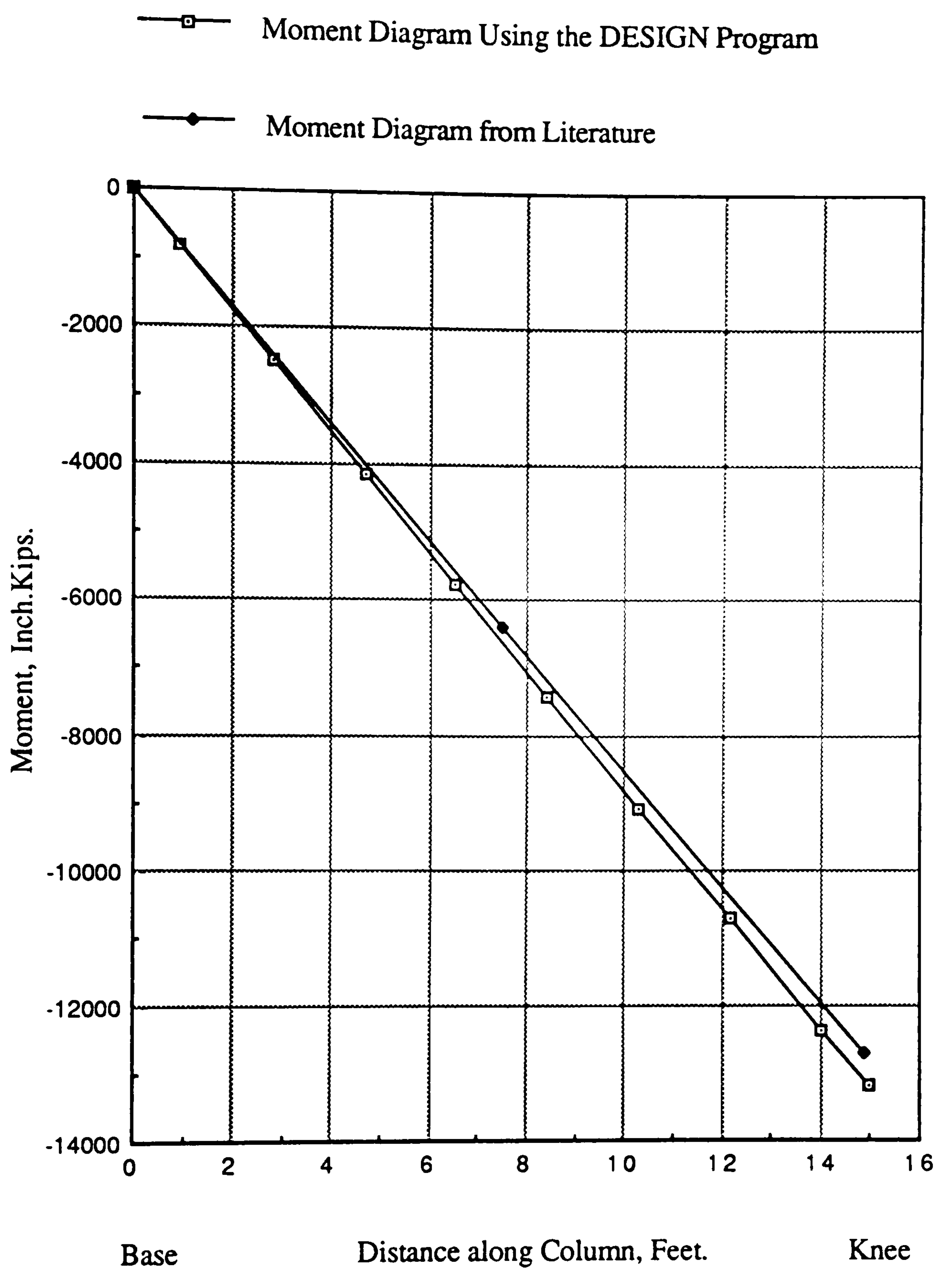

Figure B1 Validation of Column Moments using the DESIGN Program.
(Roof Slope .5/12. Ref. Lee et al. 1981 pp. 94)

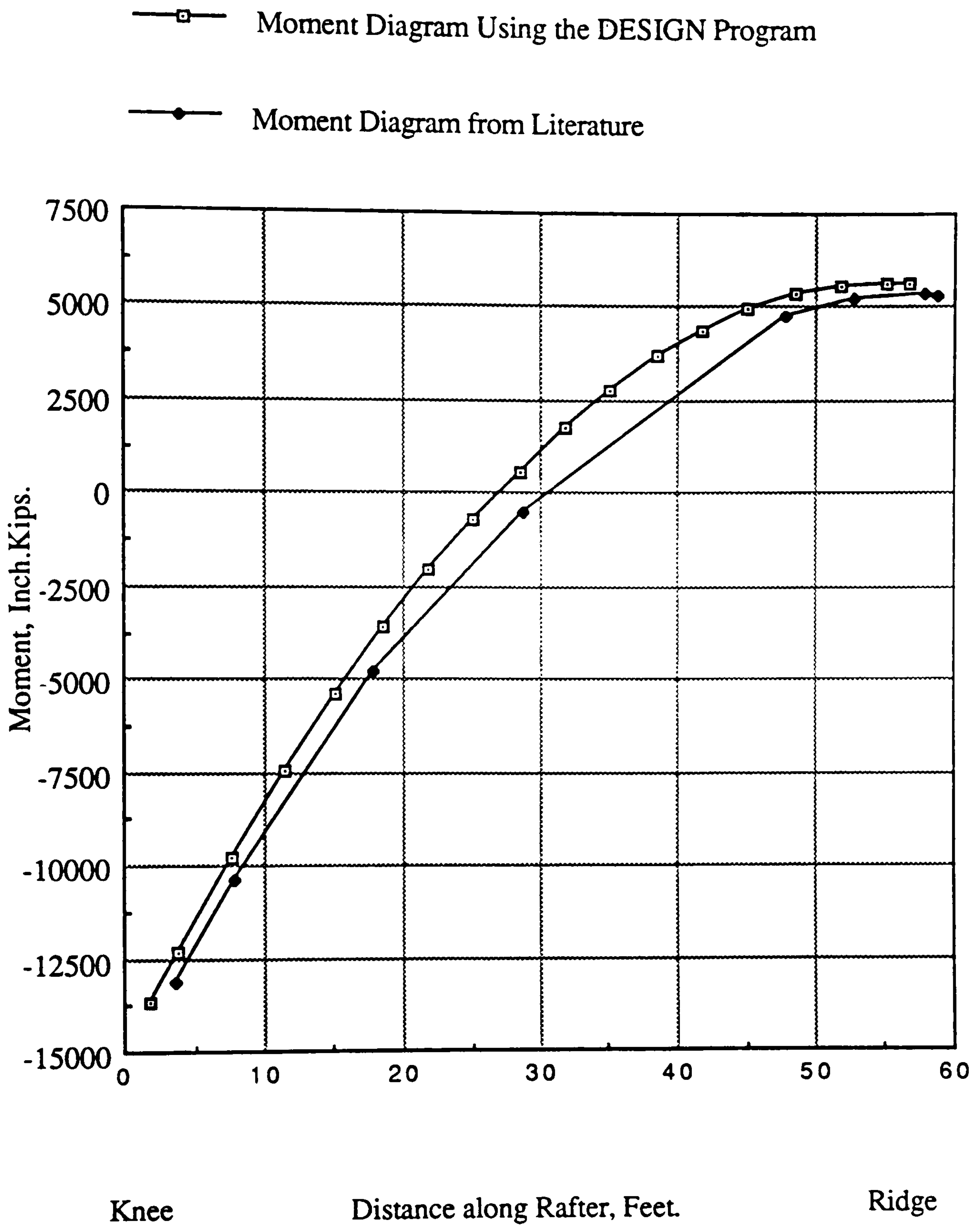

Figure B2 Validation of Rafter Moments using the DESIGN Program.
(Roof Slope .5/12. Ref. Lee et al. 1981 pp. 94)

APPENDIX C

AISC DESIGN FORMULAS

AISC equations (D3-1) and (D3-2) as given in the AISC Appendix D for the design of web tapered members were used to compute the allowable extreme fibre tension and compression stresses due to bending. Further the bending stress in the compression flange was reduced as per AISC equation (1.10-5).

Allowable Shearing Stresses were computed using AISC equation (1.10-1), further checks were made to satisfy the d/t requirements using equation (1.10-2).

Allowable compressive stresses were computed using eqn (D2-1) and (D2-2). Combination of axial and bending stresses satisfied the requirement of section (D4-1a), (D4-1b), and (D4-2).

Curves provided by (Lee et al. 1981) for computing equivalent length parameters associated with uniform and warping torsion for web tapered members were used to compute the allowable compressive stresses for laterally unsupported sections.