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PROTECTIVE DESIGN CENTER TECHNICAL REPORT

Conventional Construction Standoff Distances of the Low and Very Low Levels of Protection IAW UFC 4-010-01

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1. PURPOSE

The Protective Design Center (PDC) was tasked by the Technical Support Working Group (TSWG) to review the three commonly used conventional construction standoff distances (CCSD) found in ATFP criteria, those being: 10-m, 25-m, and 45-m. These values are referenced in *UFC 4-010-01, "DOD Minimum Antiterrorism Standards for Buildings", 8 October 2003 and Change 1, 22 January 2007*, (MS-UFC). Meeting these standoff distances require no further blast load considerations for that facility, and standard design applies. However, window and skylight systems are required to meet all provisions of Standard 10 even if the facility meets the CCSD's.

The goal of this review is to perform a series of calculations to develop new standoff distances based on the structural analysis of standard building components used for DoD inhabited and primary gathering facilities. The following codes were used in this analysis: CEDAW, SBEDS, HazL, WinGARD and BICADS.

2. BACKGROUND

Historically, the explosive safety communities used scaled range criteria to establish safe standoff distances for facilities located close to explosive storage areas. This approach works well for large quantities of explosives at large ranges, when the loading duration approaches that of a quasi-static load. Originally, ATFP practice incorporated scaled range standoff distances due to the limited testing data available then. Recent testing shows this approach is too conservative, as conventionally designed building components respond within the dynamic response regime. Analysis for this loading condition requires a dynamic analysis design using an SBEDS type analysis of those components.

For years now, buildings that met the conventional construction standoff distance, as defined in UFC 4-010-01, Tables B-1 and B-2, did not require an additional blast load analysis of those structures. Therefore, the walls, doors, windows, and roofs considered conventional construction, are included in the design of these structures. Typically, the designer analyzes the blast load capacities of conventionally constructed systems. Windows of facilities within the CCSD must include all provisions of Standard 10, therefore, requiring the use of a minimum one-quarter inch nominal thickness window with a 30-mil PVB interlayer.

Installations that knowingly do not have the CCSD have recognized the extra cost applied to each project. Often times the design team would relocate the facility and reduce the CCSD during the planning or design phase, not recognizing the ATFP cost impacts of this decision. Those costs then become an unfunded project requirement. Both scenarios have significant cost impacts to building projects. Installations recognizing these costs changed their base ATFP standards now include extra design analysis and acknowledge the higher project costs.

Installations unable to meet the CCSD's require an analysis to review the standoff distance requirements for their building components. Revising CCSD's can provide project design cost savings. In order to revise the CCSD, designers perform calculations for each charge weight as defined in the MS-UFC and for each component. This study analyzed wall, roof, and window components. This study did not analyze of doors, as they only need to swing outward per the MS-UFC.

1. Recent full-scale structural blast testing shows the MS-UFC standoff distance criteria are too conservative when using scaled ranges.
2. After changing the level of protection definitions to damage boundaries, the scaled range approach became antiquated, and the CCSD's need revision. New calculations to set the revised CCSD's require the use of damage boundary criteria.

3. STUDY PROCEDURE

To account for the new damage level definitions, several factors are considered:

1. Building component response
    a. Primary components
    b. Secondary structural components
    c. Non-structural components
2. Hazard level of glazing
3. Human injuries based on component debris and blast in-fill pressures

Based on the damage level definitions from MS-UFC, Appendix A, Table 2-1, the recommended standoff distances selected were the greatest values calculated, that don't exceed the damage definitions for building component damage, glazing hazards, and human injuries.

4. APPROACH

1. Use CEDAW to predict the component flexural resistance for each damage boundary definition. Axial loads to wall components are not considered. The response limits used in the CEDAW analysis are the same as those in SBEDS.
2. Use HazL to predict window glazing hazard levels based on the analytical modeling method. The UK model used a limited amount of data during its development and was not scalable to large combinations of glazing lite shapes and thicknesses.
3. Use WinGARD to predict charge weight to standoff relationships.
4. Use BICADS to predict human injuries from flying building debris.
5. Review the structural response of common building components used in the construction of inhabited and primary gathering facilities.
6. Review the current practice using scaled ranges of 10-m, 25-m, and 45-m.
7. Review the structural component response based on the damage level definitions based on component type.
8. Review the hazards from failed windows.
9. Review the human injuries created by debris from failed building components.
10. Provide the Security Engineering Working Group (SEWG) these findings.
11. Provide a set of graphs capturing the findings analyzed.

Referenced codes are SBEDS (*Single degree of freedom Blast Effects Design Spreadsheet*), CEDAW (*Component Explosive Damage Assessment Workbook*), HAZL (*window fragment Hazard Level analysis*), WinGARD (Window Glazing Analysis Response and Design), and BICADS (*Building Injury Calculator and DatabaseS*).

Software Limits. The current CCSD's relate the scaled ranges of 11, 18 (or 21.6 for 1 scenario), 24, 30, and 40 to judge the buildings level of protection. The ranges used are based on the broad definitions found in *Estimating Damage to Structures from Terrorist Bombs, Field Operations Guide, ETL 1110-4-495, which is very similar to the DoD Ammunition and Explosive Safety Standards, DoD 5154.45, 23 June 1980*. To minimize the effects of close-in charges and to assure a plane wave condition, the minimum scale range of three is used. The lowest practical scaled range limit within SBEDS, CEDAW and HAZL is three, and does not check wall breaching. Normally, breaching is a concern when the scaled range is between one and four. This study assumes plane waves, flexural response controls, and no load averaging.

CEDAW analysis used to predict building component performance assumed the wall decoupled analysis from the building framing system. Therefore, the analysis calculated support reactions, but no response of the building frame.

1. Response limits were taken from the *PDC-TR-06-08, "Single Degree of Freedom Response Limits for Antiterrorism Design", October 2006*. Component damage boundary limits are based available test data.
    a. Damage boundary limits, Primary Components are between *B1* and *B2* for a *Low Level of Protection* and between *B2* and *B3* for a *Very Low Level of Protection*. This study did not use damage boundary B1 since building cladding damage is the primary focus.
    b. Damage boundary limits, Secondary Component is between *B2* and *B3* for a *Low Level of Protection* and between *B3* and *B4* for a *Very Low Level of Protection*.
2. Lightweight construction consists of wood and steel stud walls. These systems have seen limited testing, and their response limits are subject to engineering judgment. Exterior Foam Insulation System (EFIS), and non-composite brick veneer in-fill systems are studied.
    a. The assumptions and results for the wood stud walls are in Appendix B.
    b. The assumptions and results for the steel stud walls are in Appendix C.
3. Heavy construction consists of concrete and masonry walls. Four walls are studied unreinforced walls, lightly reinforced walls, moderately reinforced, and heavily reinforced. The reinforcement ratios are defined in Appendix D. Only grouted cells contain reinforcing. All walls studied as in-fill construction. Axial loads are not included in analysis work, conservative.
    a. Analysis assumptions and results for masonry walls found in Appendix D, concrete walls in Appendix E and European block walls in Appendix F.
    b. In-fill panels are secondary components. Only the flexural response of the reinforced masonry walls considered.
4. Lightweight wall façades are metal panels with girts and considered compliant systems. Maximum span for metal panels are the spacing of girts.
    a. Assumptions and results of metal panels are found in Appendix G
    b. Assumptions and results of girts are found in Appendix H.

HazL analyzed windows for the positive phase blast load. Analysis included three sizes of windows and three lay-ups. Charge Weight-Standoff charts were created using HazL and WinGARD. Charge Weight-Standoff charts produced by the analytic model in HazL compared well to WinGARD. The analytic model in HazL worked better then the UK Laminated model. WinGARD default values were used.

1. Three window sizes were from eight to 96 square feet in area.
2. Analyzed were two-laminated windows, one with a 30–mil PVB interlayer, and the other with a 60-mil PVB interlayer. Both layups included two 1/8" pieces of glass. The one IGU analyzed had an outer pane of quarter inch annealed monolithic glass, with a half inch airspace, and an inner pane of a nominal quarter inch of laminated annealed glass (two-piece of one eighth inch annealed glass) with a 30–mil PVB interlayer.
3. The analysis uses positive phase blast waves.
4. Laminated glass rarely breaks out of the frame and falls within one meter into the room. This study calculated the maximum standoff distance associated with just keeping the glass in the frame. This is a very low hazard rating or a low level of protection. In addition, this analysis assumes the window frame and its anchorages are adequate to hold the glazing in the frame and the frame to the wall.
5. Assumptions and results from the window glass analysis are in Appendix I.

BICADS analysis estimates human injuries to occupants after blast damage occurs, and uses *P-i* methodology to make those calculations. Four injury categories are determined for each occupant based on their location within the building, and the construction materials used in the building envelope. The blast locations and charge weights match the MS-UFC criteria and then reduced to match the component standoff distances used in this study. Occupants locations within the building are based on typical population densities and percentage of occupants located along perimeter walls or within the central core areas. A human injury analysis calculates the percentages of occupants injured and their level of injuries for each floor level. This analysis counts injuries based on fly-in of damaged building components, window debris, and failure of interior non-structural components, direct blast loads, or progressive collapse. BICADS builds an injury statistical database for each type of injury by floor level to create an injuries report. Taken from this report are serious injuries and the onset of human fatalities data.

1. Injury data from terrorist bombing events, accidental explosions, explosive tests, simple engineering models, and engineering judgment is included in BICADS injury calculations.
2. The very low level of protection standoff distances selected is associated with 1% fatalities limit and 1% standoff, which is the onset of serious injuries.
3. BICADS analyses results are in Appendix J.

5. FINDINGS

There are several take away ideas that came out of this study. Consider several factors when recalculating the CCSD based on damage level definitions.

1. Building component response did not match the assumed damage levels assumed by the MS-UFC because scaled range standoff distances do not assure a plane wave loading. The analysis did not show close-in effects would dominate, as walls had holes blown through them, and a flexural response did not occur. Therefore, walls did not fail in a pure flexural response. Apparently, the wall vented very rapidly and reduced the total load on the wall through the sacrificial action of the wall elements.

a. The current CCSDs are not bad for installations that can meet those spacing requirements. Some reduction in standoff distances to match the right hand side of the flexural response analysis is justified.
b. Engineering judgment to set wooden wall standoff distances is required as current testing indicates these structures perform well at greatly reduced ranges. The local breaching failure and the enhanced construction of the wall probably saved the rest of the wall from failing in flexure. However, it does indicate a large locally damaged area may be more acceptable than previously thought. However, quantifying the extent of that damage with out a higher order analysis (discrete FEA) is many times impractical. Consistently predicting this type of a response using an SDOF analysis is impractical.
c. Tempering the results of the wooden stud walls used in the D-Ra tests is justified, as the construction system used does not match standard wood framed systems. Jambs studs along the window opening stacked five to seven studs along each jamb where normal construction would have used two, one king stud running the full height of the wall and one jack, or double to support the window header. These walls used more sheathing nails than normally specified by the building codes. This study found no data to support the use of these extras. Shear capacity of the stud and its connections controls the walls strength and resistance. Without an axial load on those connections to provide some end friction those connections, fail rapidly. Axially load effects studs as they go into double curvature, which increases the flexural resistance of the stud wall system. The connections respond as if they had fixed ends and ultimately fail in horizontal shear along the length of the stud.
d. Brick veneer stud walls have more mass and these systems perform better for blast-loaded systems. Therefore, a heavy façade such as brick is more desirable than a lighter façade like Exterior Foam Insulating System (EFIS) on stud wall systems. However, by applying the EFIS over a steel sheet substrate the system will perform better, as the steel plate substrate adds considerable mass to the system.
e. Steel stud walls have a great potential for being a successful blast wall, but the weak link in the system is the stud to track connections. Using two self-tapping screws to attach the flanges of the stud and track while adequate for high wind loads are inadequate for high pressure impulsive blast loads. Recent FEA connection modeling shows end connection details are available to enable the full development of its web shear strength. Therefore, steel stud walls can perform at a similar level as wood stud walls. Axial load on stud walls lead to higher wall resistance as seen in field-testing and from SBEDS analysis. Increased performance of top slip tracks occurs when backed by a four to six inch bearing angle, or bearing the wall along the edge of a floor slab. Therefore, with improvements to the conventional connections used in steel stud wall construction, it is possible to achieve the resistance developed by wood stud walls.

f. This study reviewed unreinforced masonry and European block as nonbearing walls. The addition of axial load would add to the walls flexural resistance, but was ignored to remain conservative.
g. To develop plastic hinges in structural elements, strong connections are required. Therefore, ductile structural systems perform very well for the blast loadings found in the MS-UFC, and those standoff distances are conservative. This study focused on less ductile and more brittle systems and found greater variability in the level of protection provided by these systems. In many cases, they are not conservative.

2. The strength of the window systems or the maximum glazing resistance controls the hazard levels for glazed window and door systems. Glazing thickness, aspect ratio, glass breaking strength, post break membrane strength and membrane strain limits control the maximum flexural resistance of glazed systems, glazing, window frame and wall stiffener. ASTM E 1300, HazL analytical or WinGARD models calculate the maximum glazing resistance used to design the window frame and anchorage loads. Using HazL or WinGARD allow the designer to utilize fully the glazing's PVB membrane capacity. This minimizes the glazing thickness and wall stiffener loads. Whereas, ASTM E 1300 requires the use of thicker glazing and higher system loads for anchorage design. Dynamic design using HazL WinGARD and SBEDS leads to an optimized wall system. Calculate the wall response using SBEDS and the dynamic properties of the window and wall system. The end shear reactions of the wall stiffener are reacted into the upper and lower structural floor slabs using the full flexural resistance of the wall stiffener and the Equivalent Static Load approach.
    a. Wingard and HazL do a good job of predicting the Glazing Hazard Level if the proper input values is used. Recent testing demonstrates the UK Laminate Model is unconservative. The HazL analytic and WinGARD models matched well with recent window testing, both matched well to blast loaded test data.
    b. The IGU window studied showed that a medium level of protection is a good design selection as the glazing cracks and stays in the frame. Experience and studies confirm there is a "cookie-cutter" effect as the typical glazing system will either stay in the frame as a minimal hazard, or fly across the room as a high hazard. It is very difficult to get a window to fail and fall within one meter of the wall and have a low hazard rating.
    c. Selecting the glass thickness based on the dynamic response of the windows as calculated by HazL and WinGARD yields a significantly different piece of glass when compared to the ASTM E 1300 approach. These codes account for the nonlinearity of glass, and the ultimate membrane resistance of the PVB interlayer.
    d. While, this study only looked at the positive phase duration of the blast load, WinGARD and HazL can also handle the negative phase of the blast load. The negative phase loading can be beneficial if it reaches the window as it is hitting its peak displacement when the negative phase loading comes onto the window. Should the window reach its peak displacement before the positive blast load is complete the window is pressure sensitive and the blast impulse is not a key design factor. However, when the window reaches, its peak displacement after the positive phase blast load has passed and before

the negative phase is complete, net impulse controls the window response. With this scenario, the window could fail rebounding to the outside of the structure.

e. BICADS did not predict significant serious human injuries based on component debris and blast in-fill pressures.

6. RECOMMENDATIONS

While computer codes give the criteria writers some general guidance on the use of standard standoff distance, they are not all inclusive. This study demonstrates the bounding limits with the charge weight to standoff distances found in the attached appendices. The best use of these charts is to select the right hand set of curves as the CCSD for the MS-UFC. This study has shown that the confidence level in some of the output values is questionable and some engineering judgment is required. For instance, the design standoff for wood stud walls is well within the radius of major human injury while the BICADS model predicts very few injuries. This occurs because the BICADS database is looking at thrown debris and ignores the blast shock wave that can produce significant injuries to the eardrums and lungs before the debris injures or kills people. The recommended CCSD's from this study are summarized in Table 1. which shows the recommended standoff distances after considering the effects of construction response, window hazard levels, and human injuries and fatalities from debris and blast effects. The largest standoffs distance then controlled the recommended value shown. Below are summarized standoff distance recommendations that came out of this study: Set standoff distances based on the type of construction, similar to that as shown in Table 1. The SEWG should consider the following recommendations:

1. Using Component damage instead of fixed standoff distances.
2. Conservatively use the positive phase loading.
3. Require windows use a dynamic design procedure, but not less than the existing MS-UFC.
4. Use the UFC 3-340-02 spherical charge and incident pressure to set the Human injury limits. Base standoff distances on thresholds for eardrum rupture, lung damage, and lethality, as air blast shocks are more damaging to humans then shown by debris throw calculations.
5. Distinguish standoff distances based on the type of wall construction.
6. Consider retrofitting unreinforced masonry wall for major renovation projects to existing facilities. While not allowed for new projects, many retrofit projects may benefit from wall strengthening systems currently available.
7. Steel stud construction could benefit by using better connection details.

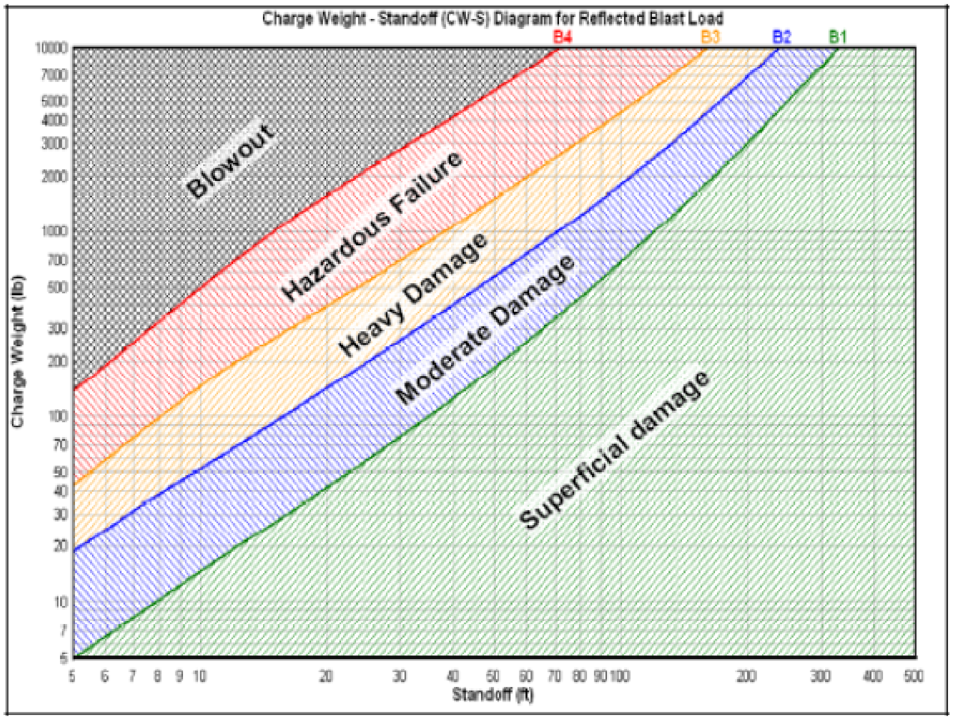


Figure 1 - Charge Weigh-Standoff Diagram Showing Component Damage

Component Damage Level	Relationship to Response Limits
Blowout	Response greater than B4.
Hazardous Failure	Response between B3 and B4
Heavy Damage	Response between B2 and B3.
Moderate Damage	Response between B1 and B2.
Superficial Damage	Response is less than B1.

Figure 2 - Component Damage Levels Relationship to Response Limits

Table 1 - Recommended Standoff Distances

Wall Type	Charge Weight I				Charge Weight II			
	Load Bearing B2	Load Bearing B3	Non Load Bearing B3	Non Load Bearing B4	Load Bearing B2	Load Bearing B3	Non Load Bearing B3	Non Load Bearing B4
Existing UFC Baseline	148’	82’	148’	82’	82’	33’	82’	33’
Wood Studs – Brick Veneer	104’	104'	78’	65’	36’	36'	33' (3)	33' (3)
Wood Studs – EFIS	207’	207'	163’	140’	85’	85'	66’	55’
Metal Studs – Brick Veneer	186	108’	206' (2)	186' (2)	74’	42’	82' (2)	74' (2)
Metal Studs – EFIS	360’	206’	419' (2)	361' (2)	150’	85’	167' (2)	150' (2)
Metal Panels	n/a(1)	n/a(1)	152’	108’	n/a(1)	n/a(1)	55’	39’
Girts	n/a(1)	n/a(1)	114’	58’	n/a(1)	n/a(1)	33' (3)	33' (3)
Reinforced Concrete	67’	67'	33' (3)	33' (3)	33' (3)	33' (3)	33' (3)	33' (3)
Unreinforced Masonry	262’	262'	124’	34’	80’	80'	33' (3)	33' (3)
Reinforced Masonry	86’	86'	33' (3)	33' (3)	33' (3)	33' (3)	33' (3)	33' (3)
European Block	164’	164'	59’	33' (3)	38’	38'	33' (3)	33' (3)

1: Metal panels and girts are not considered primary members
2: Non-load bearing steel studs are assumed to have slip-track connections
3: Analysis indicates stand-off less than 33' minimum, which is prohibited

Appendix A - Expanded Table 2-1 from UFC 4-010-01

Level of Protection	Potential Building Performance	Building Component Damage			Potential Door and Glazing Hazards[10]	Potential Injury
		Primary Structural Components[2]	Secondary Structural Components[3]	Non-structural Components[4]		
Below AT standards[1]	Severe damage. Progressive collapse likely. Space in and around damaged area will be unusable.	Hazardous damage[6]	Blowout[5]	Blowout[5]	Doors and windows will fail catastrophically and result in lethal hazards. (High hazard rating)	Majority of personnel in collapse region suffer fatalities. Potential fatalities in areas outside of collapsed area likely.
Very Low	Heavy damage - Onset of structural collapse, but progressive collapse is unlikely. Space in and around damaged area will be unusable.	Heavy Damage[7]	Hazardous damage[6]	Hazardous damage[6]	Glazing will fracture, come out of the frame, and is likely to be propelled into the building, with the potential to cause serious injuries. (Low hazard rating) Doors may be propelled into rooms, presenting serious hazards.	Majority of personnel in damaged area suffer serious injuries with a potential for fatalities. Personnel in areas outside damaged area will experience minor to moderate injuries.
Low	Moderate damage – Building damage will not be economically repairable. Progressive collapse will not occur. Space in and around damaged area will be unusable.	Moderate Damage[8]	Heavy Damage[7]	Heavy Damage[7]	Glazing will fracture, potentially come out of the frame, but at a reduced velocity, does not present a significant injury hazard. (Very low hazard rating) Doors may fail, but they will rebound out of their frames, presenting minimal hazards.	Majority of personnel in damaged area suffer minor to moderate injuries with the potential for a few serious injuries, but fatalities are unlikely.. Personnel in areas outside damaged areas will potentially experience a minor to moderate injuries.
Medium	Minor damage – Building damage will be economically repairable. Space in and around damaged area can be used and will be fully functional after cleanup and repairs.	Superficial Damage[9]	Moderate Damage[8]	Moderate Damage[8]	Glazing will fracture, remain in the frame and results in a minimal hazard consisting of glass dust and slivers. (Minimal hazard rating) Doors will stay in frames, but will not be reusable.	Personnel in damaged area potentially suffer minor to moderate injuries, , but fatalities are unlikely. Personnel in areas outside damaged areas will potentially experience superficial injuries.
High	Minimal damage. No permanent deformations. The facility will be immediately operable.	Superficial Damage[9]	Superficial Damage[9]	Superficial Damage[9]	Glazing will not break. (No hazard rating) Doors will be reusable.	Only superficial injuries are likely.

Notes:
1. This is not a level of protection, and should never be a design goal. It only defines a realm of more severe structural response, and may provide useful information in some cases.
2. Primary structural component can be identified as a member whose loss would affect a number of other components supported by that member and whose loss could potentially affect the overall structural stability of the building in the area of loss. Examples include columns and girders and other primary framing components directly or in-directly supporting other structural or non-structural members. Also, any load-bearing structural components.
3. Secondary structural component = Non-load bearing infill wall components and any other structural component supported by a primary framing component.
4. Non-structural component can be identified as a member whose loss would have little effect on the overall structural stability of the building in the area of loss. Examples include interior non-load bearing walls, overhead lights, heaters, and other mechanical or architectural items attached to building structural components
5. Blowout: The component is overwhelmed by the blast load causing debris with significant velocities.
6. Hazardous Damage: The component has failed, and there is anywhere from no significant velocity of component debris to some debris with significant velocity.
7. Heavy damage: The component has not failed, but it has significant permanent deflections causing it to be unrepairable. The component is not expected to withstand the same blast load again without failing.
8. Moderate damage: The component has some permanent deflection. It is generally repairable, if necessary, although replacement may be more economical and aesthetic. The component is expected to withstand the same blast load again without failing but the end state may be a lower level of protection.
9. Superficial damage: No visible permanent damage. The component is expected to withstand the same blast load and maintain the level of protection.
10. Glass hazard levels are from ASTM F 1642.

Appendix B - Wood Studs

Wood Studs Analysis Summary:

1. Studs:
    a. 2x4 and 2x6
    b. #2 S-P-F
    c. 8 and 10 foot lengths
2. Stud Spacing's:
    a. 16 inch O.C. for both 2x4 and 2x6
    b. 24 inch O.C. for 2x6
3. Support Conditions:
    a. Simple-Simple
4. Wood Properties
    a. Wood density, γ: 30 pcf
    b. Elastic modulus, E: 1,400,000 psi
    c. Dynamic flexure yield strength, F_{db}: 4,375 psi
5. Supported Weights:
    a. EIFS: 10 psf
    b. Brick veneer: 44 psf
6. Wall Layups:
    a. W1 – 8' tall with 2x4's @ 16" O.C. and EIFS
    b. W2 – 8' tall with 2x4's @ 16" O.C. and 4" Brick Veneer
    c. W3 – 8' tall with 2x6's @ 16" O.C. and EIFS
    d. W4 – 8' tall with 2x6's @ 16" O.C. and 4" Brick Veneer
    e. W5 – 8' tall with 2x6's @ 24" O.C. and EIFS
    f. W6 – 8' tall with 2x6's @ 24" O.C. and 4" Brick Veneer
    g. W7 – 10' tall with 2x4's @ 16" O.C. and EIFS
    h. W8 – 10' tall with 2x4's @ 16" O.C. and 4" Brick Veneer
    i. W9 – 10' tall with 2x6's @ 16" O.C. and EIFS
    j. W10 – 10' tall with 2x6's @ 16" O.C. and 4" Brick Veneer
    k. W11 – 10' tall with 2x6's @ 24" O.C. and EIFS
    l. W12 – 10' tall with 2x6's @ 24" O.C. and 4" Brick Veneer

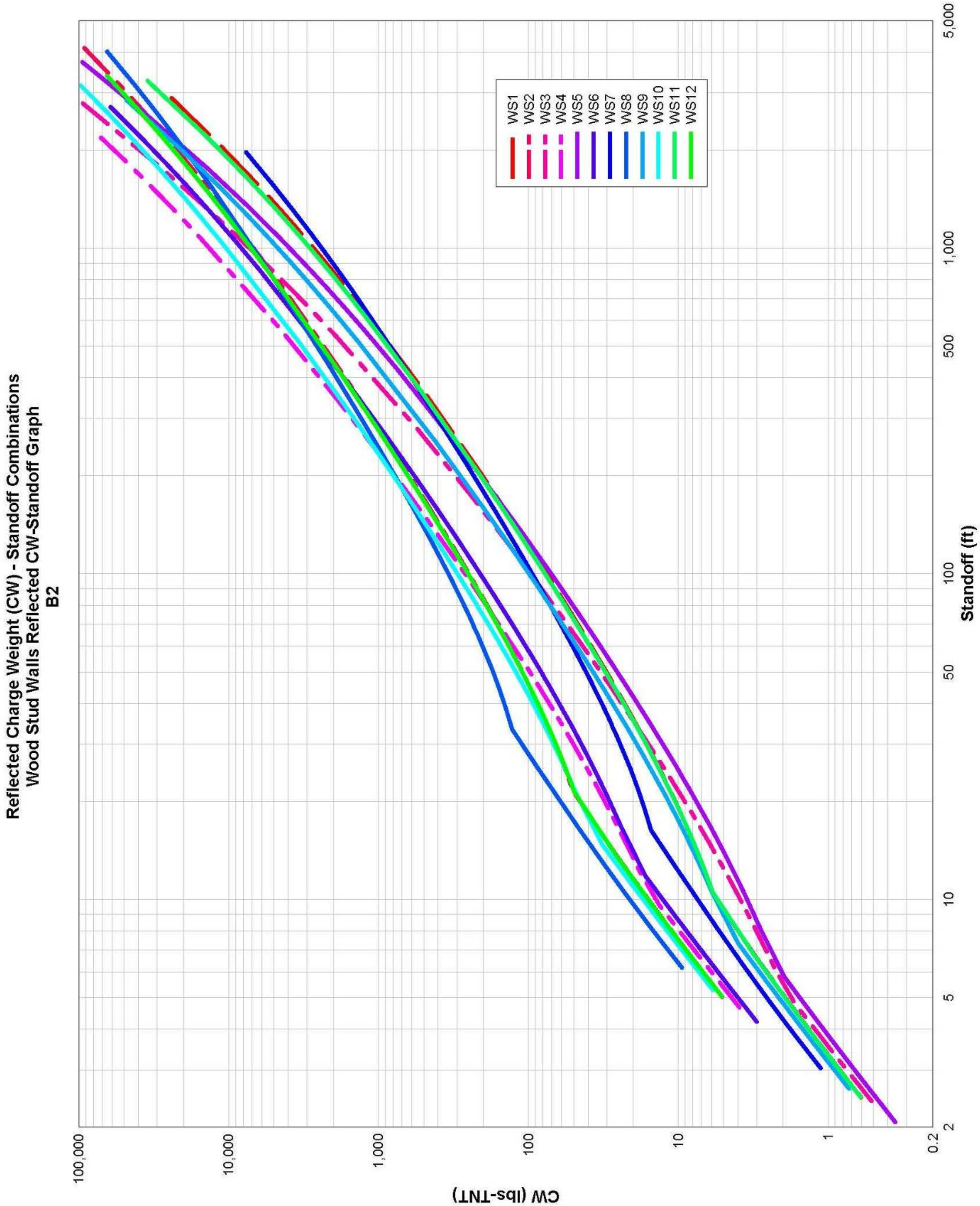


Figure B-1 – B2 Damage Curves for Woods Studs

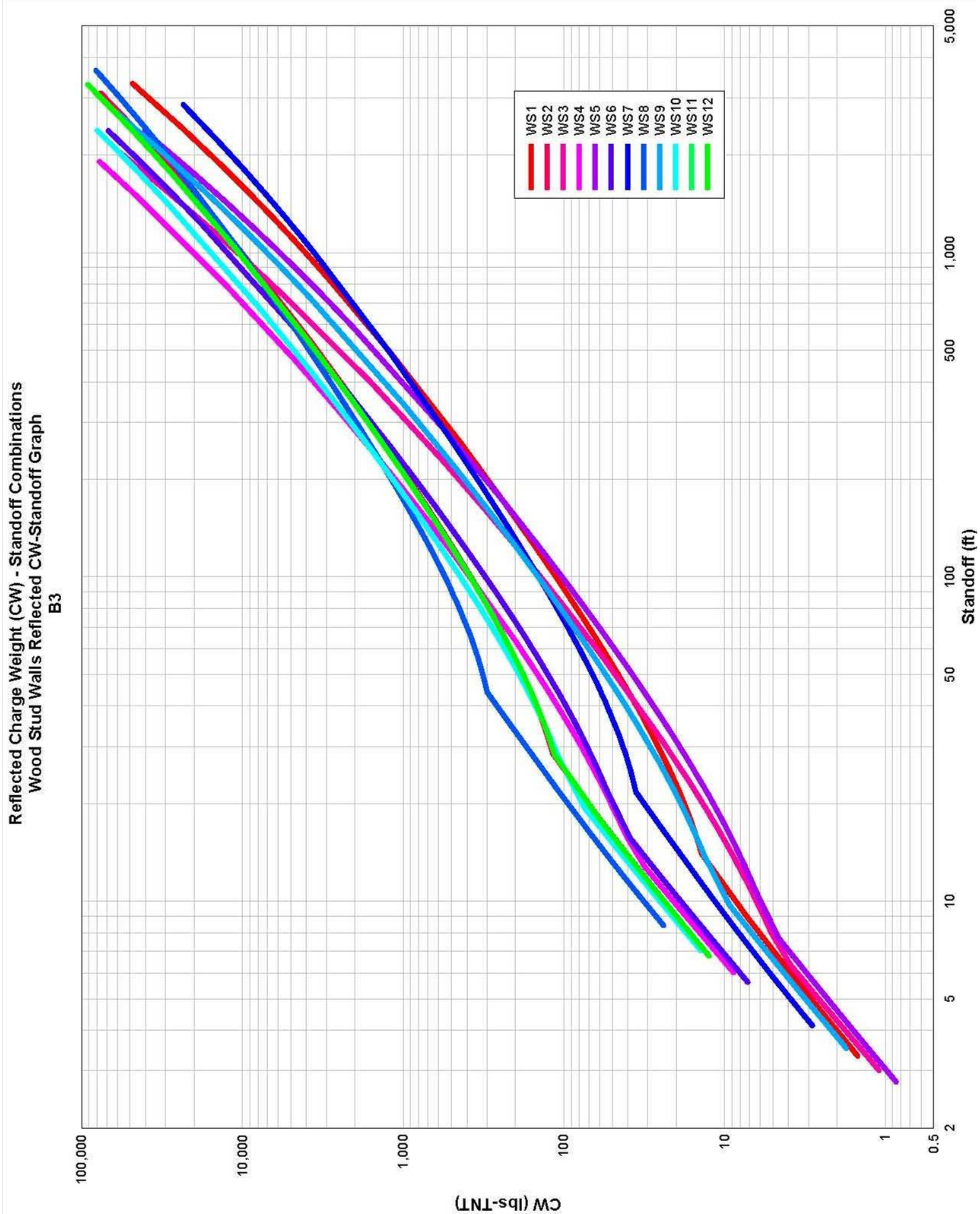


Figure B-2 – B3 Damage Curves for Wood Studs

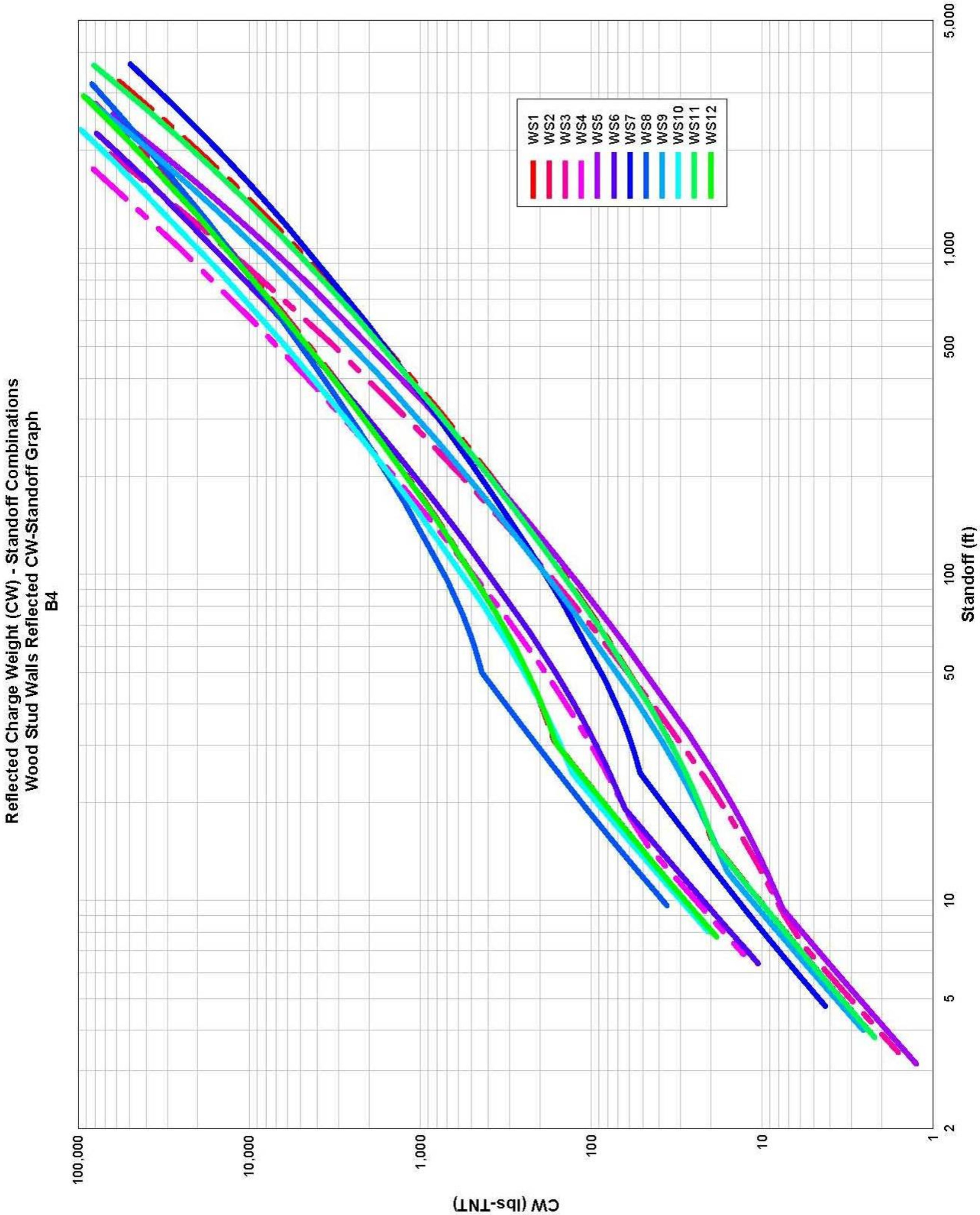


Figure B-3 – B4 Damage Curves for Wood Studs

Appendix C - Steel Studs

Steel Stud Analysis Summary:

1. Studs:
    a. 600S162-43, 600S162-54, and 600S162-68 (e.g., 6 x 1-5/8 x 68 mil)
    b. 8, 10, and 12 foot lengths
2. Stud Spacing's:
    a. 16 inch O.C.
    b. 24 inch O.C.
3. Support Conditions:
    a. Simple-Simple
4. Connections:
    a. Studs with sliding connection
    b. Studs connected top and bottom
5. Stud Properties:
    a. Yield strength, F_y: 50, 000 psi, Grade 50
    a. Elastic modulus, E: 29,000,000 psi
    b. *SIF*: 1.21
    b. *DIF*: 1.1
    c. Dynamic yield strength, F_{dy}: 66,550 psi
6. Supported Weights:
    a. EIFS: 10 psf
    b. Brick veneer: 44 psf
7. Wall Layups:
    a. MS1 – 8' tall with 600S162-43 studs @ 16" and EIFS
    b. MS2 – 8' tall with 600S162-43 studs @ 16" and 4" Brick Veneer
    c. MS3 – 8' tall with 600S162-54 studs @ 16" and EIFS
    d. MS4 – 8' tall with 600S162-54 studs @ 16" and 4" Brick Veneer
    e. MS5 – 8' tall with 600S162-68 studs @ 16" and EIFS
    f. MS6 – 8' tall with 600S162-68 studs @ 16" and 4" Brick Veneer
    g. MS7 – 8' tall with 600S162-43 studs @ 24" and EIFS
    h. MS8 – 8' tall with 600S162-43 studs @ 24" and 4" Brick Veneer
    i. MS9 – 8' tall with 600S162-54 studs @ 24" and EIFS
    j. MS10 – 8' tall with 600S162-54 studs @ 24" and 4" Brick Veneer
    k. MS11 – 8' tall with 600S162-68 studs @ 24" and EIFS
    l. MS12 – 8' tall with 600S162-68 studs @ 24" and 4" Brick Veneer
    m. MS13 – 10' tall with 600S162-43 studs @ 16" and EIFS
    n. MS14 – 10' tall with 600S162-43 studs @ 16" and 4" Brick Veneer
    o. MS15 – 10' tall with 600S162-54 studs @ 16" and EIFS
    p. MS16 – 10' tall with 600S162-54 studs @ 16" and 4" Brick Veneer
    q. MS17 – 10' tall with 600S162-68 studs @ 16" and EIFS
    r. MS18 – 10' tall with 600S162-68 studs @ 16" and 4" Brick Veneer
    s. MS19 – 10' tall with 600S162-43 studs @ 24" and EIFS
    t. MS20 – 10' tall with 600S162-43 studs @ 24" and 4" Brick Veneer

u. MS21 – 10’ tall with 600S162-54 studs @ 24” and EIFS
v. MS22 – 10’ tall with 600S162-54 studs @ 24” and 4” Brick Veneer
w. MS23 – 10’ tall with 600S162-68 studs @ 24” and EIFS
x. MS24 – 10’ tall with 600S162-68 studs @ 24” and 4” Brick Veneer
y. MS25 – 12’ tall with 600S162-43 studs @ 16” and EIFS
z. MS26 – 12’ tall with 600S162-43 studs @ 16” and 4” Brick Veneer
aa. MS27 – 12’ tall with 600S162-54 studs @ 16” and EIFS
bb. MS28 – 12’ tall with 600S162-54 studs @ 16” and 4” Brick Veneer
cc. MS29 – 12’ tall with 600S162-68 studs @ 16” and EIFS
dd. MS30 – 12’ tall with 600S162-68 studs @ 16” and 4” Brick Veneer
ee. MS31 – 12’ tall with 600S162-43 studs @ 24” and EIFS
ff. MS32 – 12’ tall with 600S162-43 studs @ 24” and 4” Brick Veneer
gg. MS33 – 12’ tall with 600S162-54 studs @ 24” and EIFS
hh. MS34 – 12’ tall with 600S162-54 studs @ 24” and 4” Brick Veneer
ii. MS35 – 12’ tall with 600S162-68 studs @ 24” and EIFS
jj. MS36 – 12’ tall with 600S162-68 studs @ 24” and 4” Brick Veneer

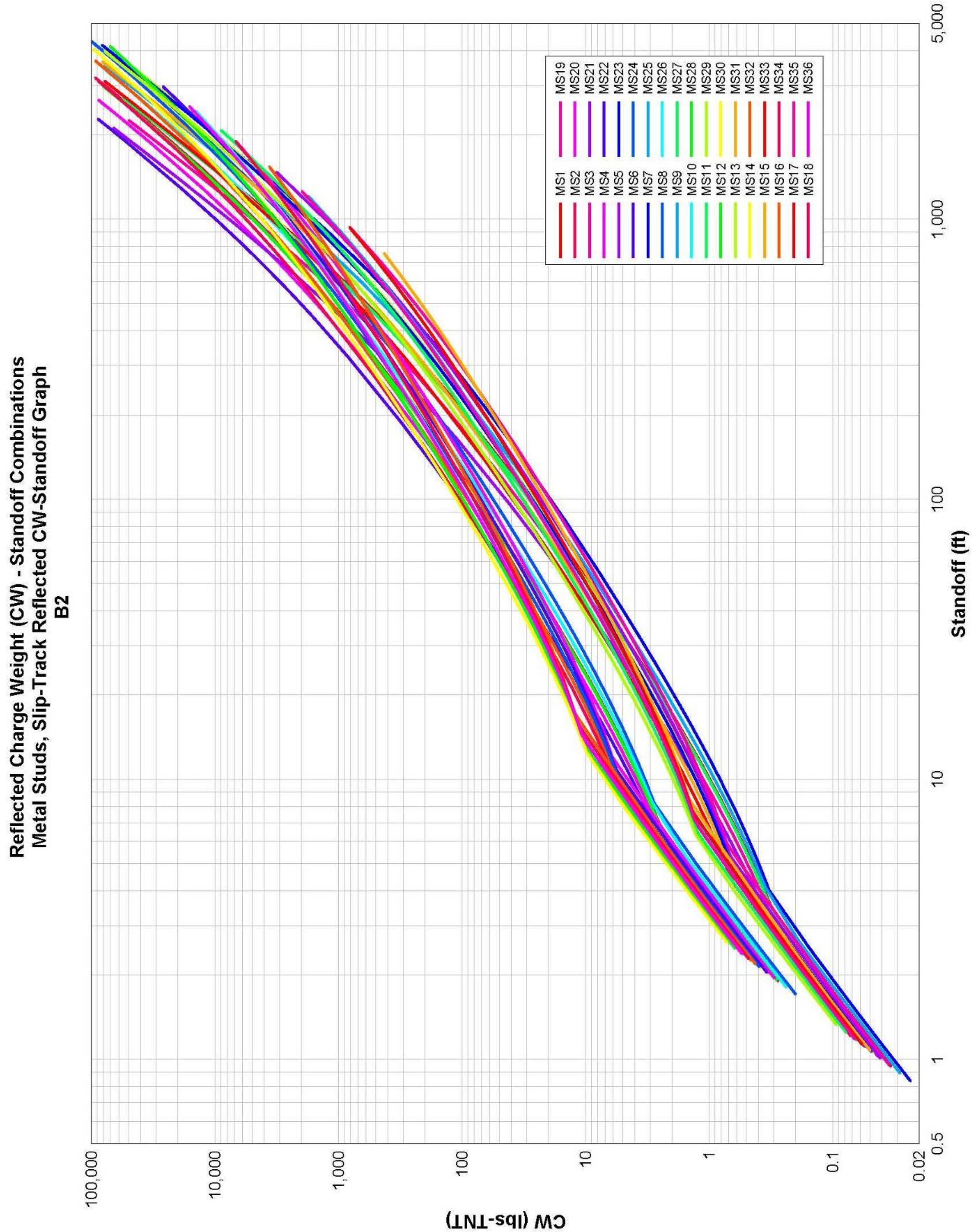


Figure C-1 – B2 Damage Curves for Metal Studs with Slip-Track Connection

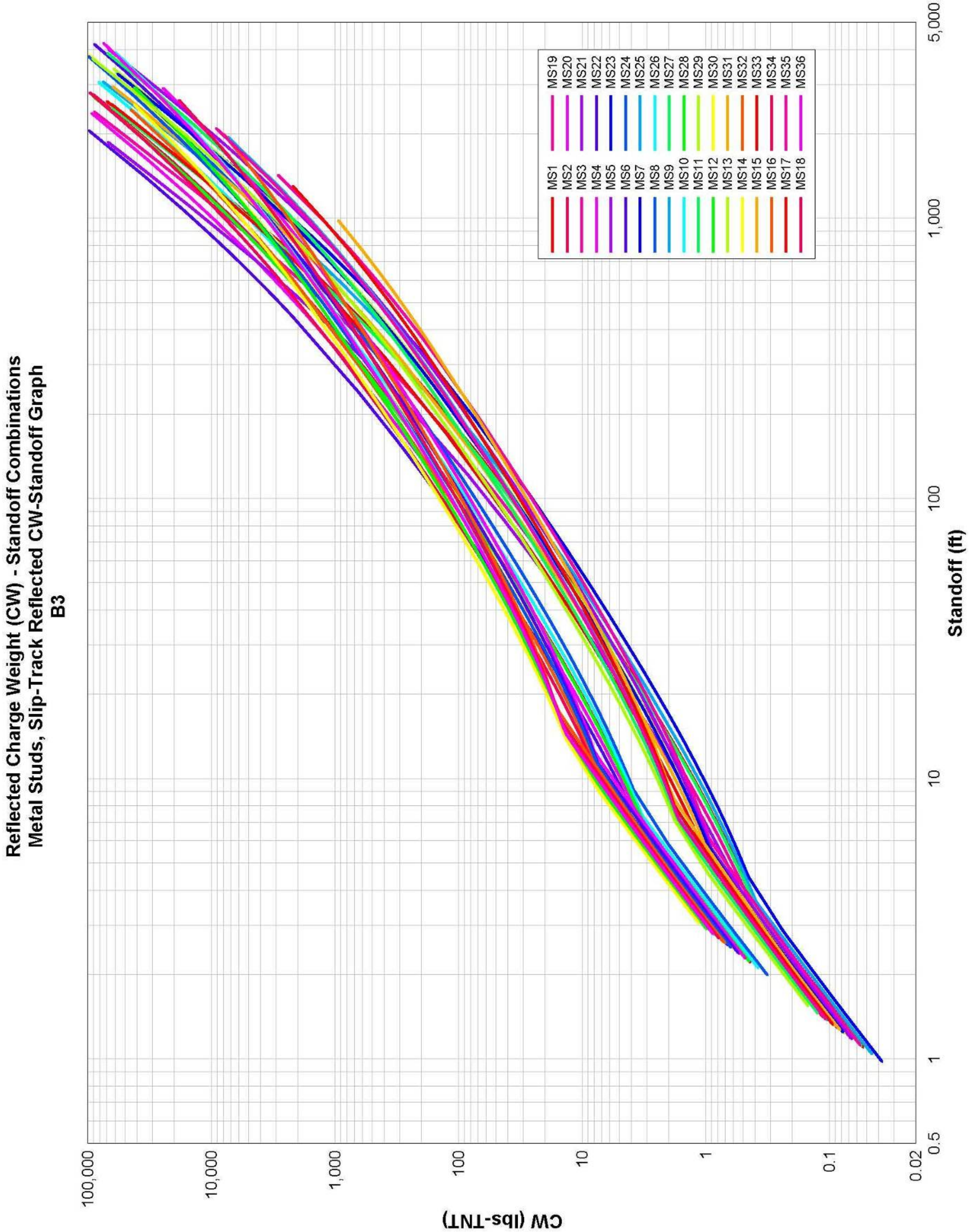


Figure C-2 – B3 Damage Curves for Metal Studs with Slip-Track Connection

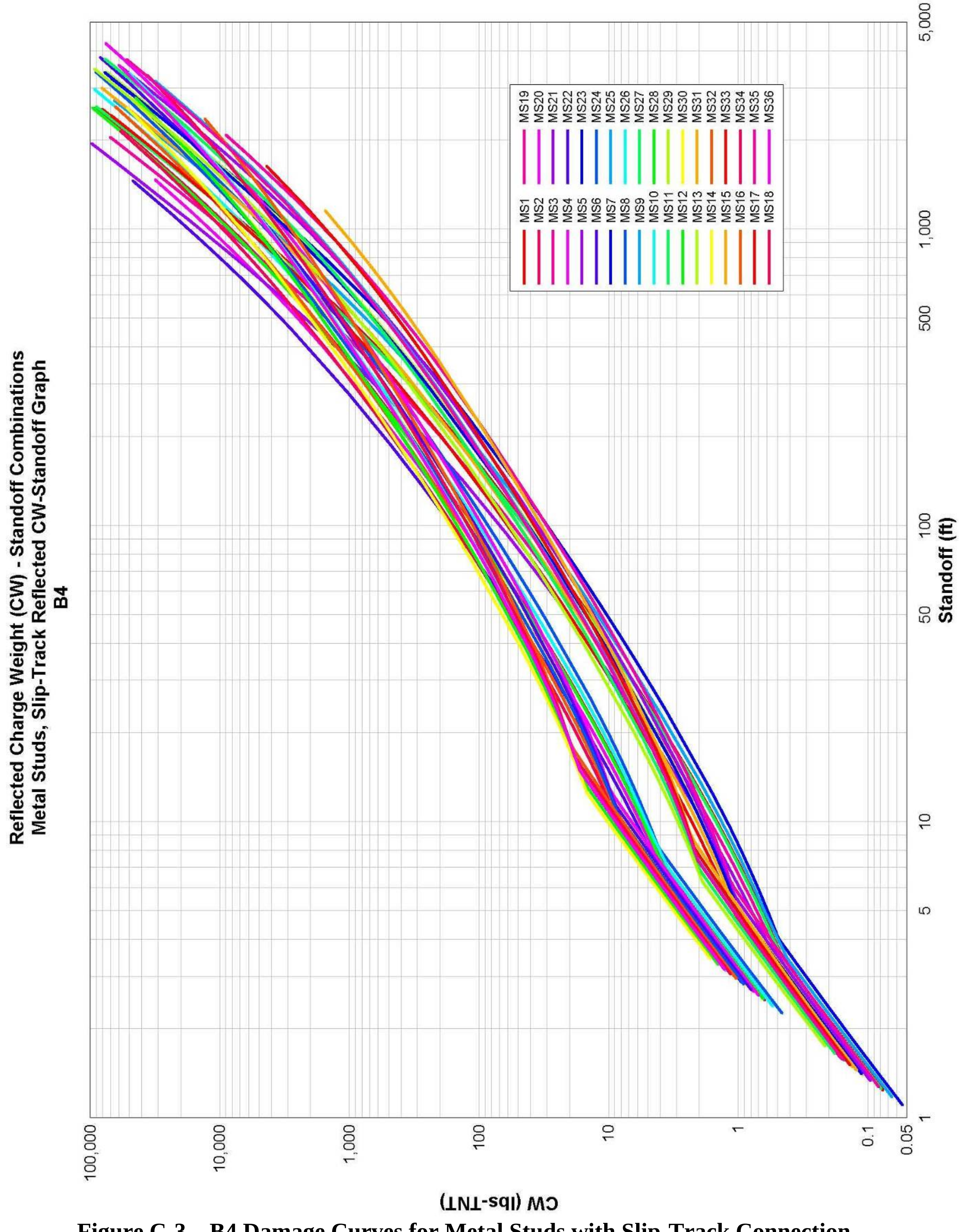


Figure C-3 – B4 Damage Curves for Metal Studs with Slip-Track Connection

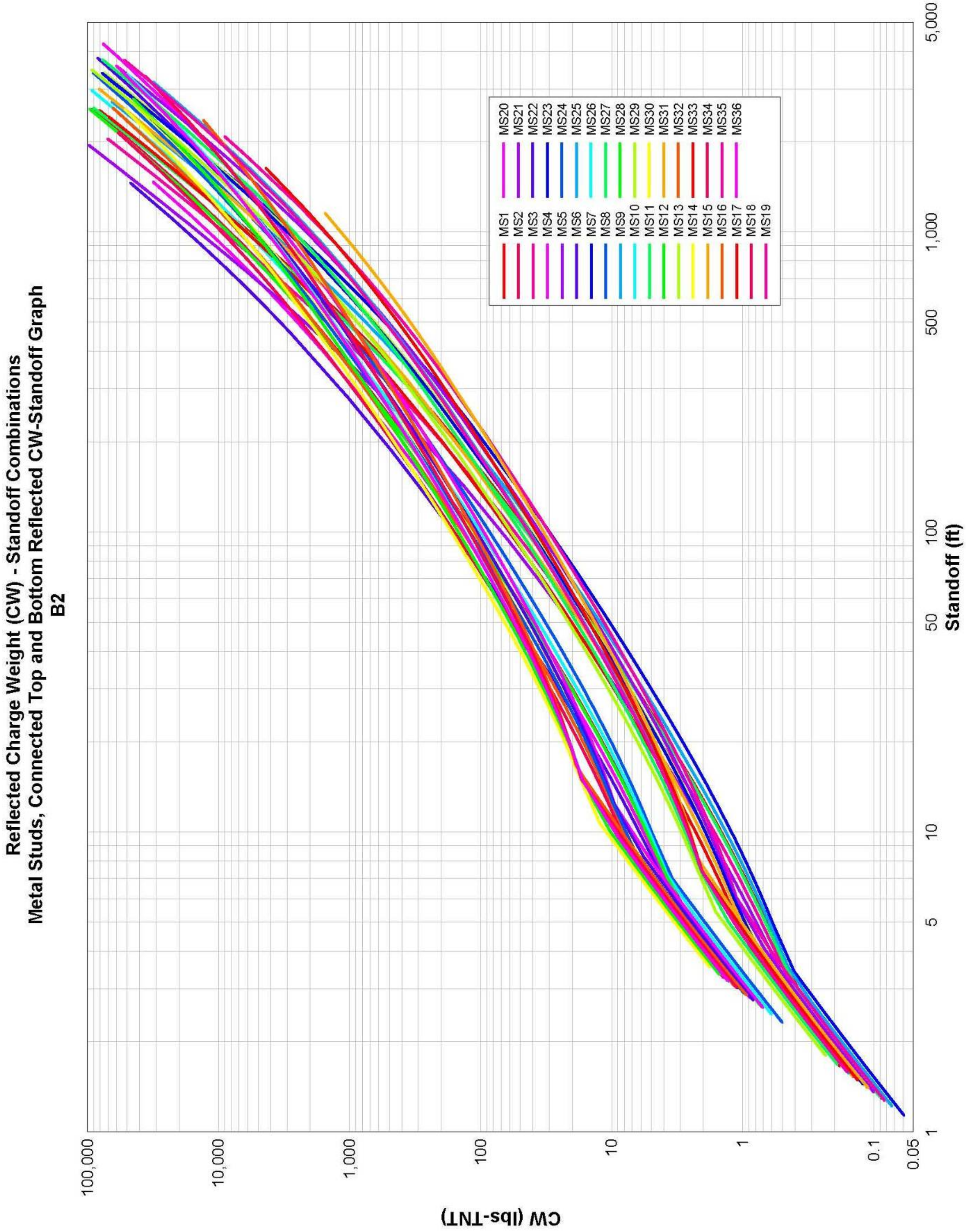


Figure C-4 – B2 Damage Curves for Metal Studs Connected Top and Bottom

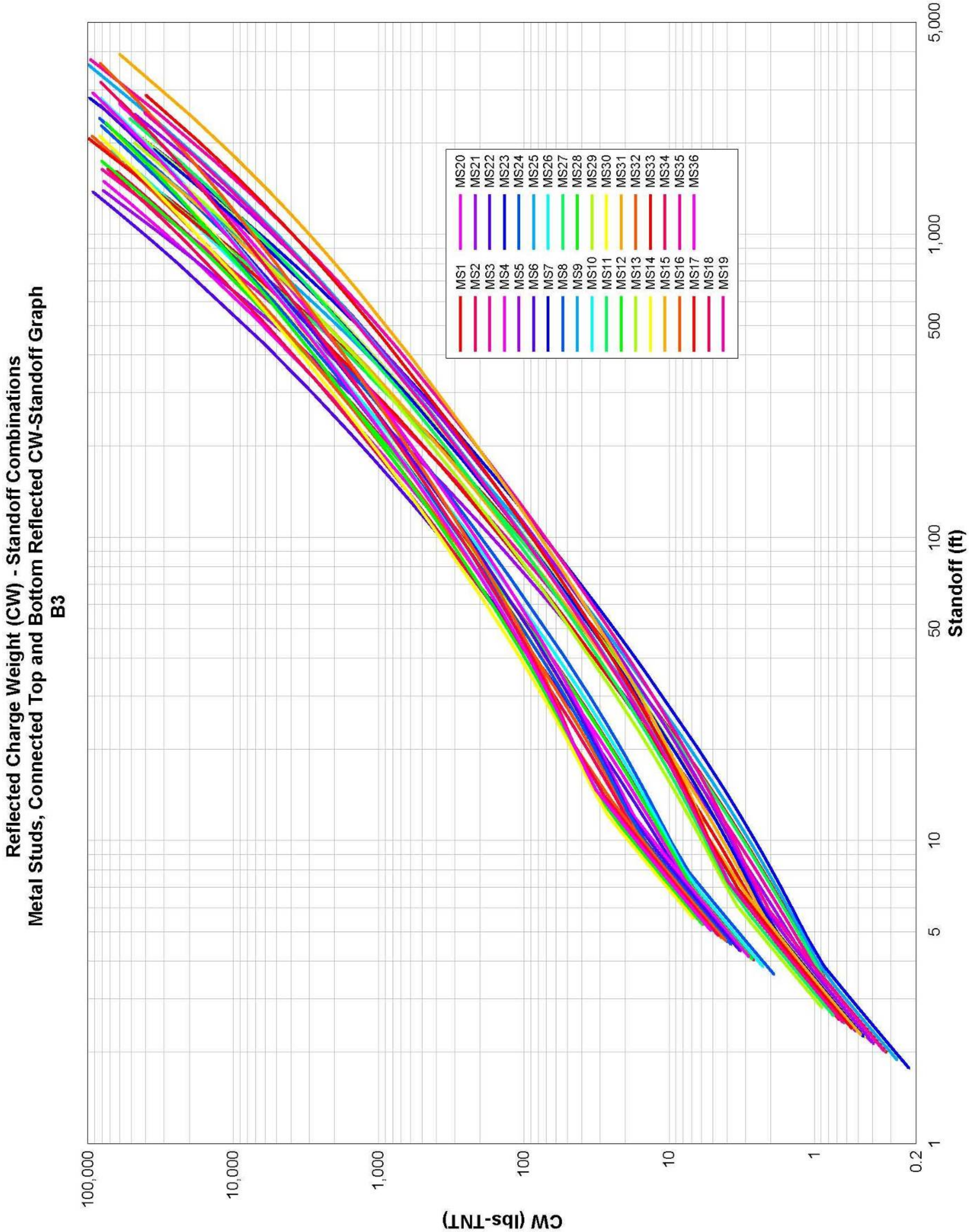


Figure C-5 – B3 Damage Curves for Metal Studs Connected Top and Bottom

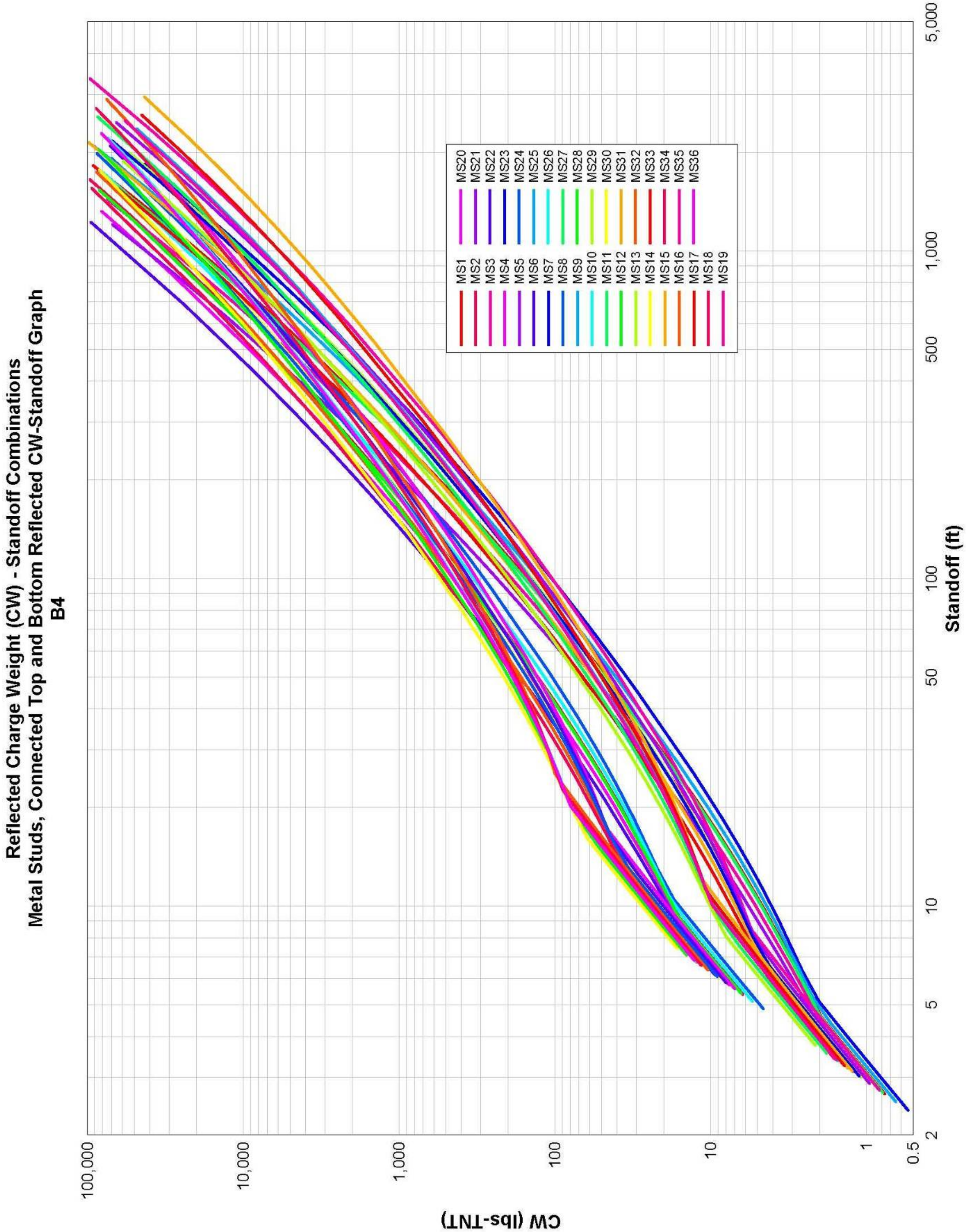


Figure C-6 – B4 Damage Curves for Metal Studs Connected Top and Bottom

Appendix D - Reinforced and Unreinforced Masonry

Masonry Analysis Summary:

1. Reinforcement:
    a. Unreinforced
    b. Light Reinforcement: 0.0005 A_g ($A_g = b\ t$, The nominal thickness of the wall is t , and the distance from the compression face to the centroid of the reinforcing steel is d .)
    c. Moderate Reinforcement: 0.0015 A_g
    d. Heavy Reinforcement: 0.0030 A_g
2. Walls:
    a. Thickness: 8, 10, and 12 inch
    b. Heights: 8, 10, 12, and 14 feet
3. Masonry Properties:
    a. Medium weight CMU, 120 pcf,
    b. Masonry compression, f'_m: 1,500 psi
    c. Reinforcement tension, F_t: 60,000 psi
    d. Masonry, *DIF*: 1.19
    e. Reinforcing bars, *DIF*: 1.17
4. Support Conditions:
    a. Simple-Simple
    b. One-way flexure
5. Tributary Widths:
    a. B_w: = 1
6. Reinforcement Location:
    a. Centered in the cell ($d = t / 2$)
7. Reinforced Masonry Wall Layups:
    a. RCMU1 – 10’ tall, 8” thick with #4’s @ 32”, 10 psf support weight
    b. RCMU2 – 10’ tall, 10” thick with #4’s @ 24”, 10 psf support weight
    c. RCMU3 – 10’ tall, 12” thick with #4’s @ 24”, 10 psf support weight
    d. RCMU4 – 12’ tall, 8” thick with #4’s @ 32”, 10 psf support weight
    e. RCMU5 – 12’ tall, 10” thick with #4’s @ 24”, 10 psf support weight
    f. RCMU6 – 12’ tall, 12” thick with #4’s @ 24”, 10 psf support weight
    g. RCMU7 – 14’ tall, 8” thick with #4’s @ 32”, 10 psf support weight
    h. RCMU8 – 14’ tall, 10” thick with #4’s @ 24”, 10 psf support weight
    i. RCMU9 – 14’ tall, 12” thick with #4’s @ 24”, 10 psf support weight
    j. RCMU10 – 10’ tall, 8” thick with #4’s @ 16”, 10 psf support weight
    k. RCMU11 – 10’ tall, 10” thick with #5’s @ 24”, 10 psf support weight
    l. RCMU12 – 10’ tall, 12” thick with #5’s @ 16”, 10 psf support weight
    m. RCMU13 – 12’ tall, 8” thick with #4’s @ 16”, 10 psf support weight
    n. RCMU14 – 12’ tall, 10” thick with #5’s @ 24”, 10 psf support weight
    o. RCMU15 – 12’ tall, 12” thick with #5’s @ 16”, 10 psf support weight
    p. RCMU16 – 14’ tall, 8” thick with #4’s @ 16”, 10 psf support weight
    q. RCMU17 – 14’ tall, 10” thick with #5’s @ 24”, 10 psf support weight
    r. RCMU18 – 14’ tall, 12” thick with #5’s @ 16”, 10 psf support weight

   s. RCMU19 – 10’ tall, 8” thick with #5’s @ 16”, 10 psf support weight
   t. RCMU20 – 10’ tall, 10” thick with #4’s @ 8”, 10 psf support weight
   u. RCMU21 – 10’ tall, 12” thick with #6’s @ 16”, 10 psf support weight
   v. RCMU22 – 12’ tall, 8” thick with #5’s @ 16”, 10 psf support weight
   w. RCMU23 – 12’ tall, 10” thick with #4’s @ 8”, 10 psf support weight
   x. RCMU24 – 12’ tall, 12” thick with #6’s @ 16”, 10 psf support weight
   y. RCMU25 – 14’ tall, 8” thick with #5’s @ 16”, 10 psf support weight
   z. RCMU26 – 14’ tall, 10” thick with #4’s @ 8”, 10 psf support weight
   aa. RCMU27 – 14’ tall, 12” thick with #6’s @ 16”, 10 psf support weight
8. Unreinforced Masonry Wall Layups:
   a. CMU1 – 8’ tall, 6” thick with 10 psf support weight
   b. CMU2 – 8’ tall, 8” thick with 10 psf support weight
   c. CMU3 – 8’ tall, 10” thick with 10 psf support weight
   d. CMU4 – 8’ tall, 12” thick with 10 psf support weight
   e. CMU5 – 10’ tall, 6” thick with 10 psf support weight
   f. CMU6 – 10’ tall, 8” thick with 10 psf support weight
   g. CMU7 – 10’ tall, 10” thick with 10 psf support weight
   h. CMU8 – 10’ tall, 12” thick with 10 psf support weight
   i. CMU9 – 12’ tall, 6” thick with 10 psf support weight
   j. CMU10 – 12’ tall, 8” thick with 10 psf support weight
   k. CMU11 – 12’ tall, 10” thick with 10 psf support weight
   l. CMU12 – 12’ tall, 12” thick with 10 psf support weight

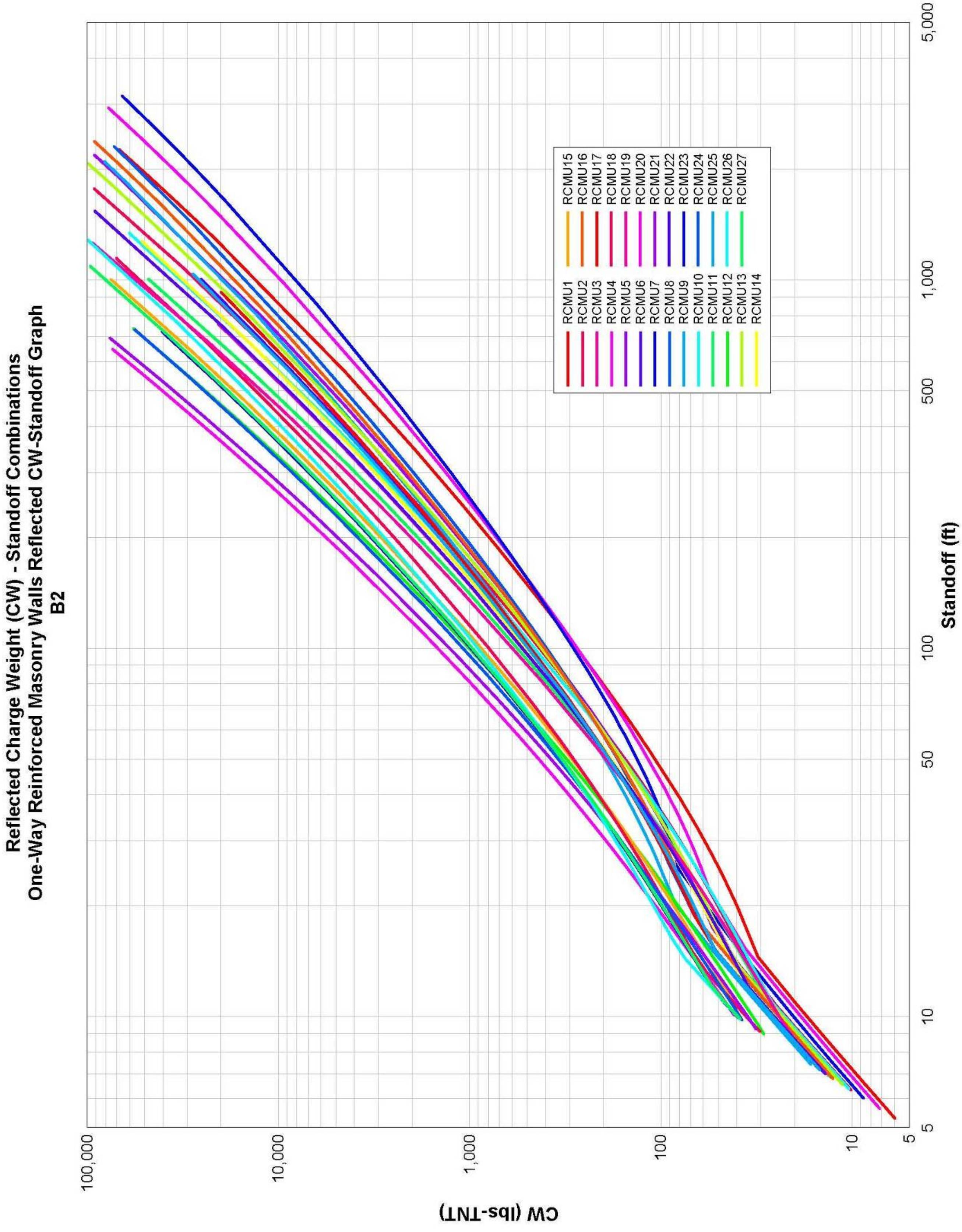


Figure D-1 – B2 Damage Curves for Reinforced Masonry

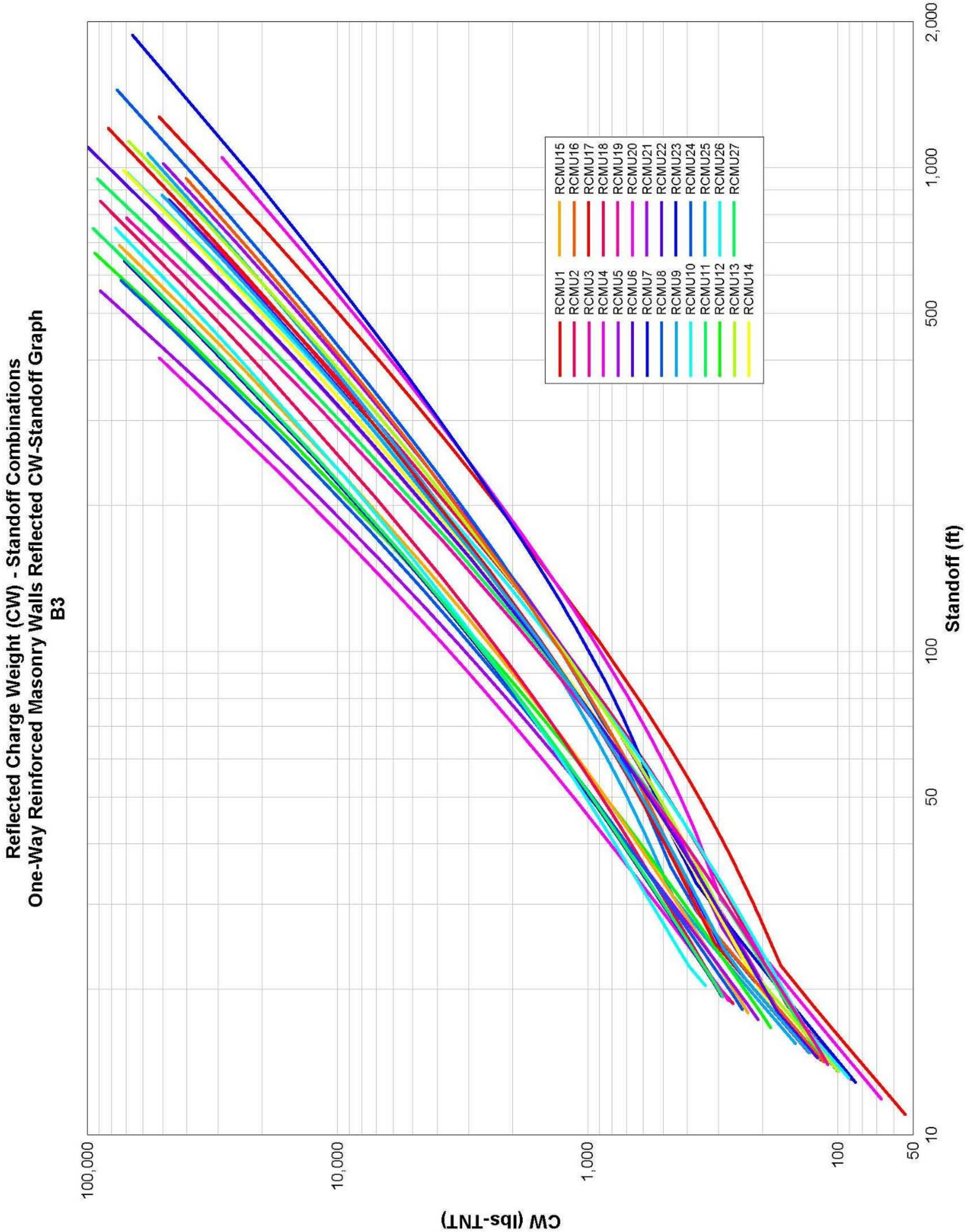


Figure D-2 – B3 Damage Curves for Reinforced Masonry

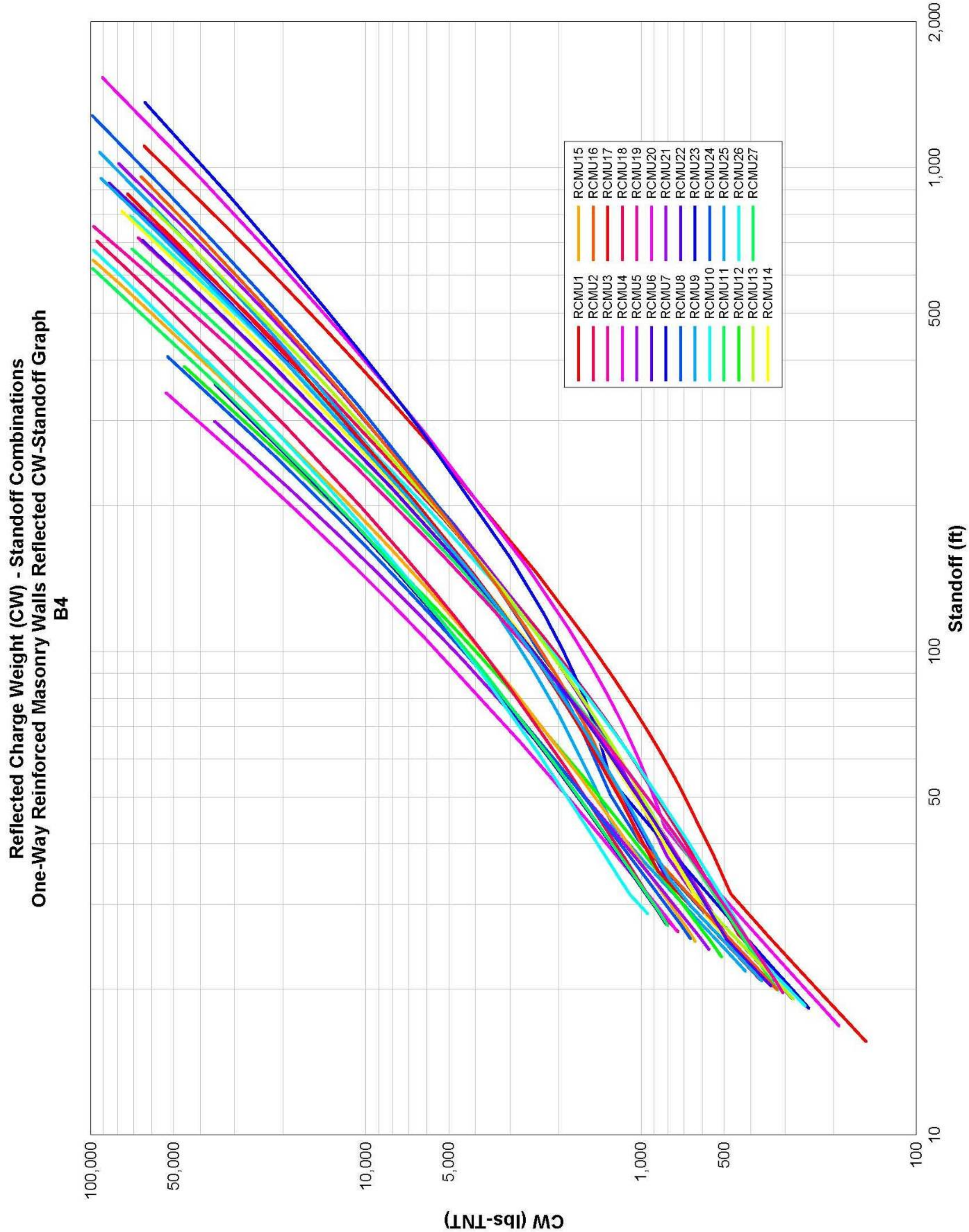


Figure D-3 – B4 Damage Curves for Reinforced Masonry

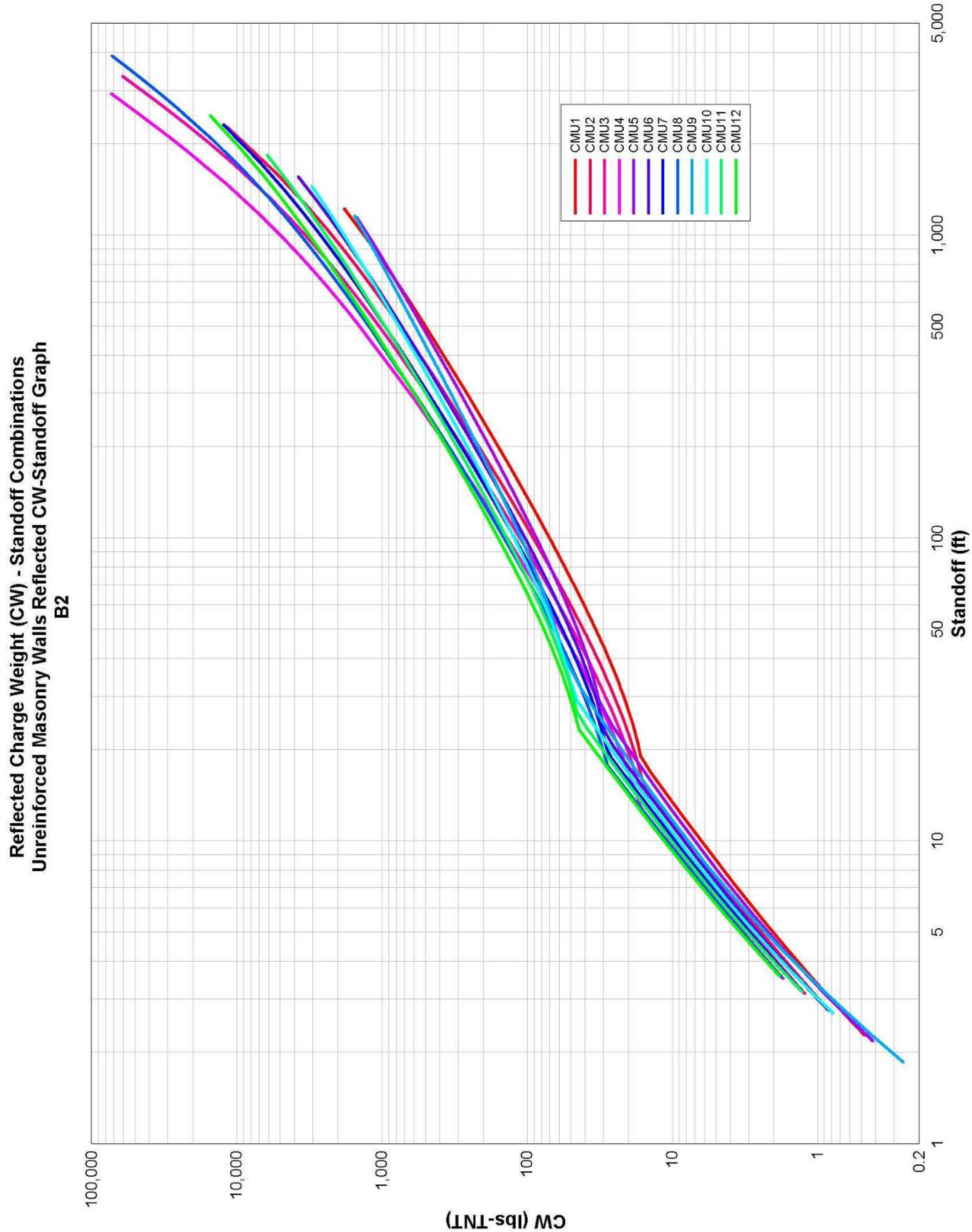


Figure D-4 – B2 Damage Curves for Unreinforced Masonry

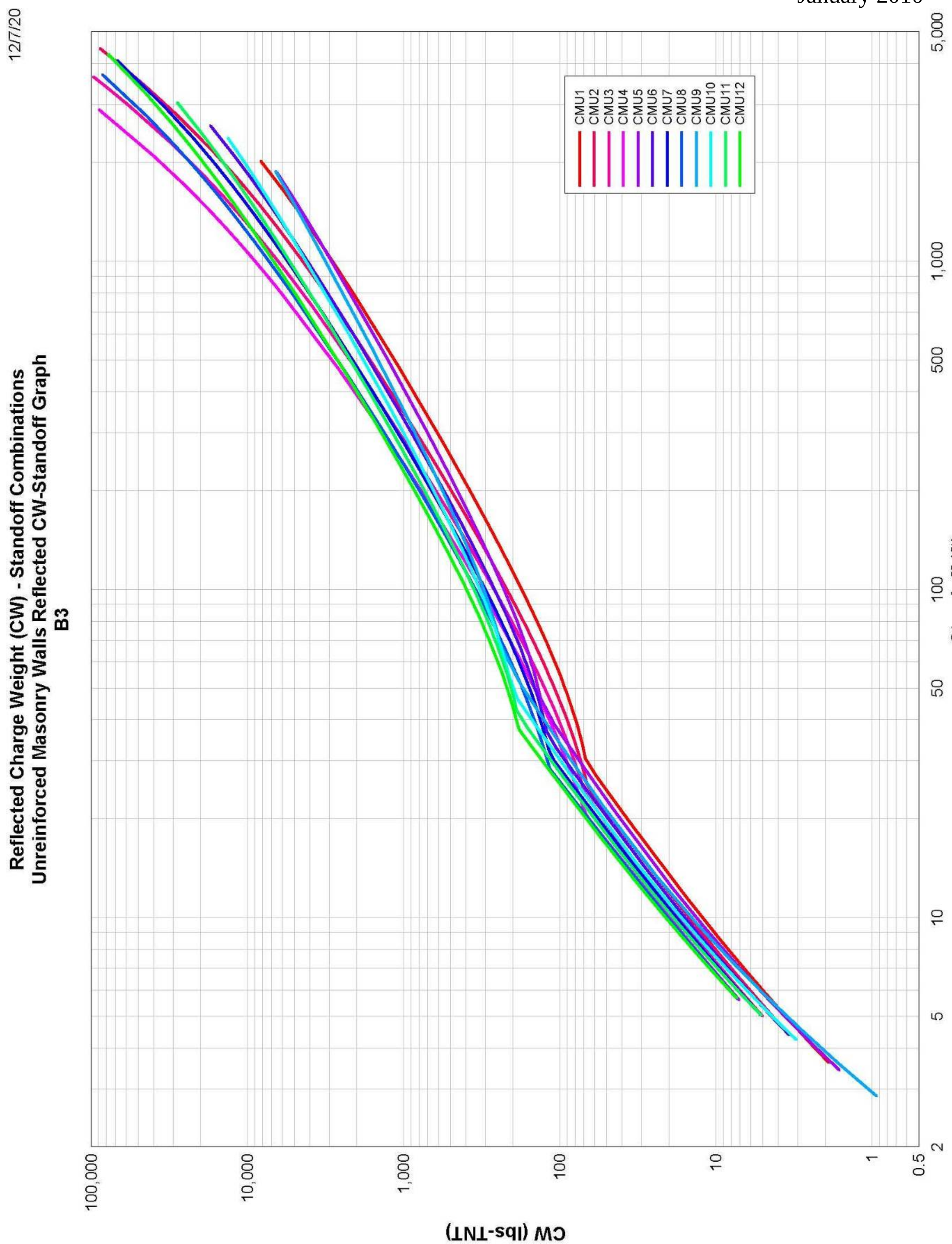


Figure D-5 – B3 Damage Curves for Unreinforced Masonry

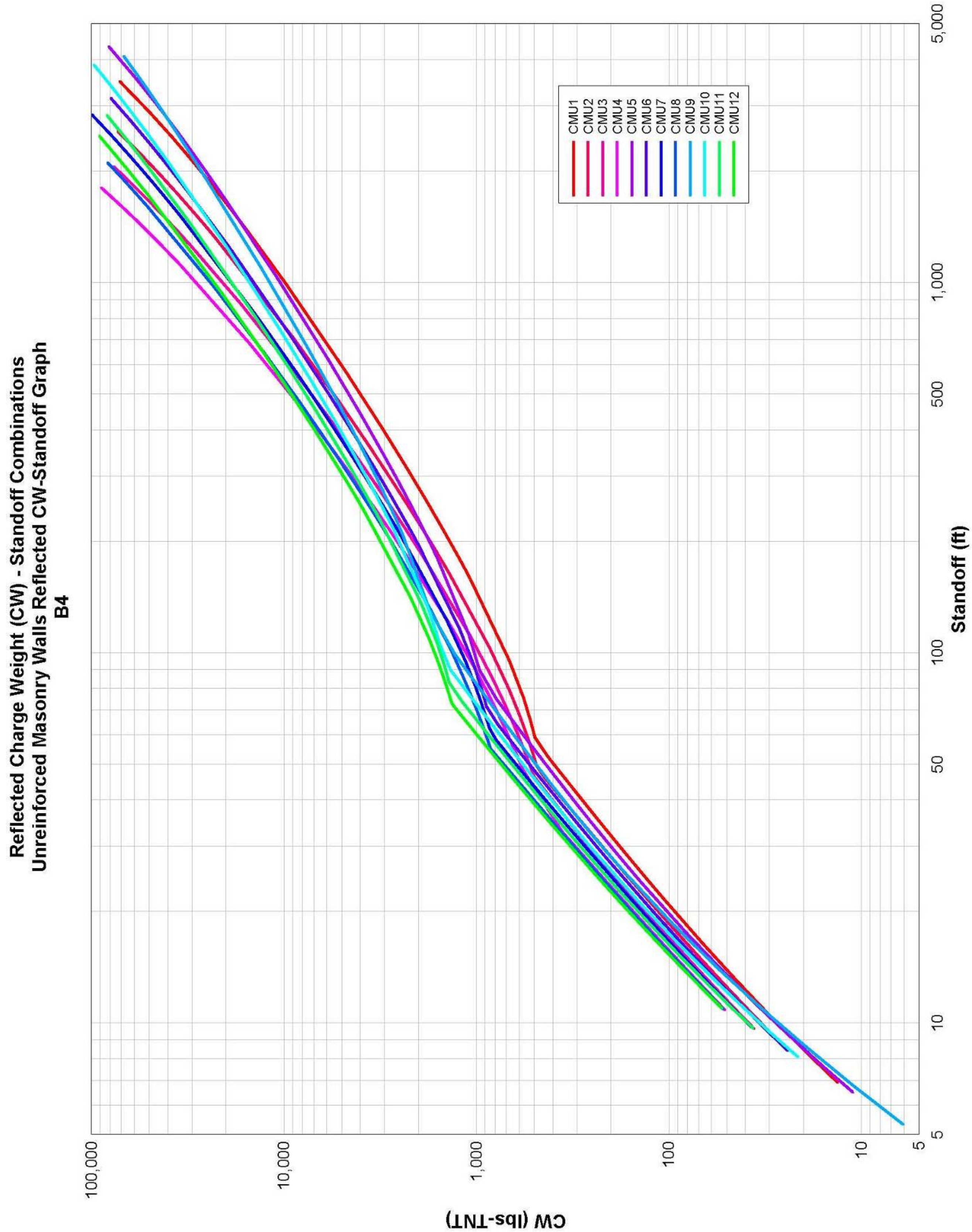


Figure D-6 – B4 Damage Curves for Unreinforced Masonry

Appendix E - Reinforced Concrete Walls

Concrete Analysis Summary:

1. Reinforcement:
    a. Light Reinforcement: 0.0015 A_g ($A_g = b\ t$, The nominal thickness of the wall is t , and the distance from the compression face to the centroid of the reinforcing steel is d .)
2. Walls:
    a. Thickness: 6 inch
    b. Heights: 12, 16 and 20 feet
3. Concrete Properties:
    a. Concrete density, 150 pcf
    b. Concrete compression, f'_c: 3,000 psi
    c. Reinforcement tension, F_t: 60,000 psi
    d. Concrete, *DIF*: 1.19
    e. Reinforcing bars, *DIF*: 1.17
4. Support Conditions:
    a. Simple-Simple
    b. One-way flexure
5. Tributary Widths:
    a. B_w. = 1.
6. Reinforcement Location:
    a. Centered in the wall ($d = t / 2$)
7. Wall Layups:
    a. RC1 – 12’ tall, 6” thick with #4’s @ 24” and 10 psf support weight
    b. RC2 – 16’ tall, 6” thick with #4’s @ 24” and 10 psf support weight
    c. RC3 – 20’ tall, 6” thick with #4’s @ 24” and 10 psf support weight

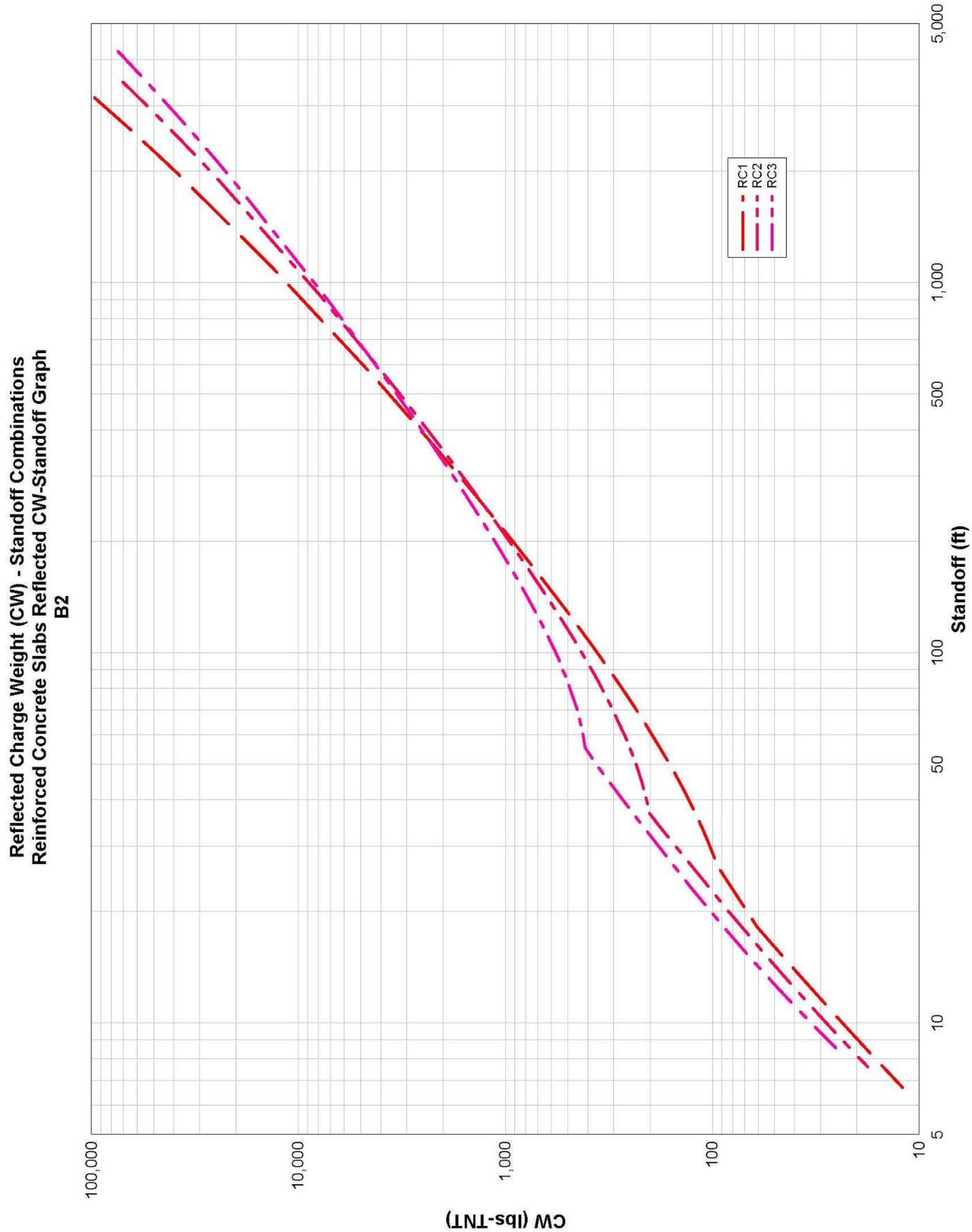


Figure E-1 – B2 Damage Curves for Reinforced Concrete

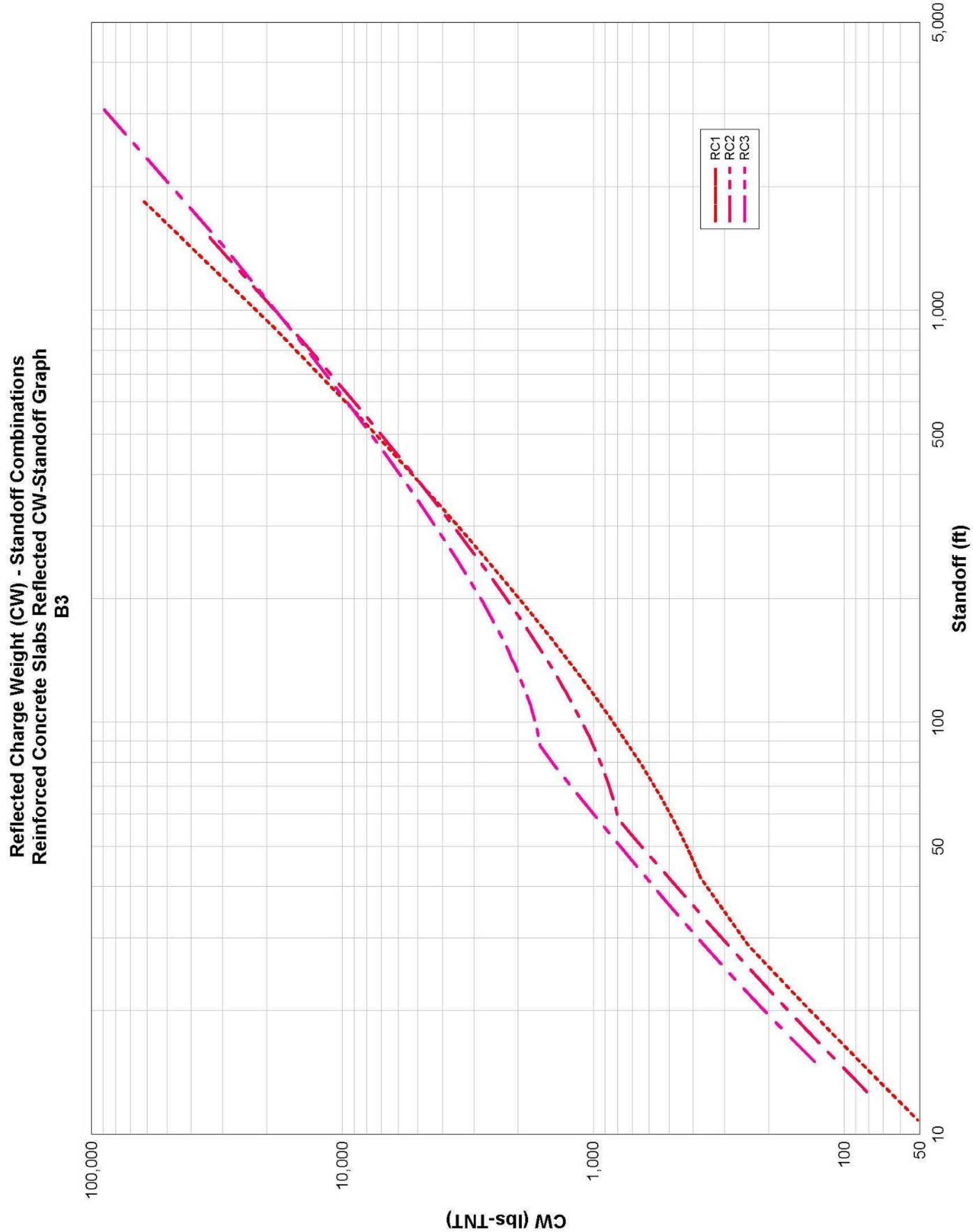


Figure E-2 – B3 Damage Curves for Reinforced Concrete

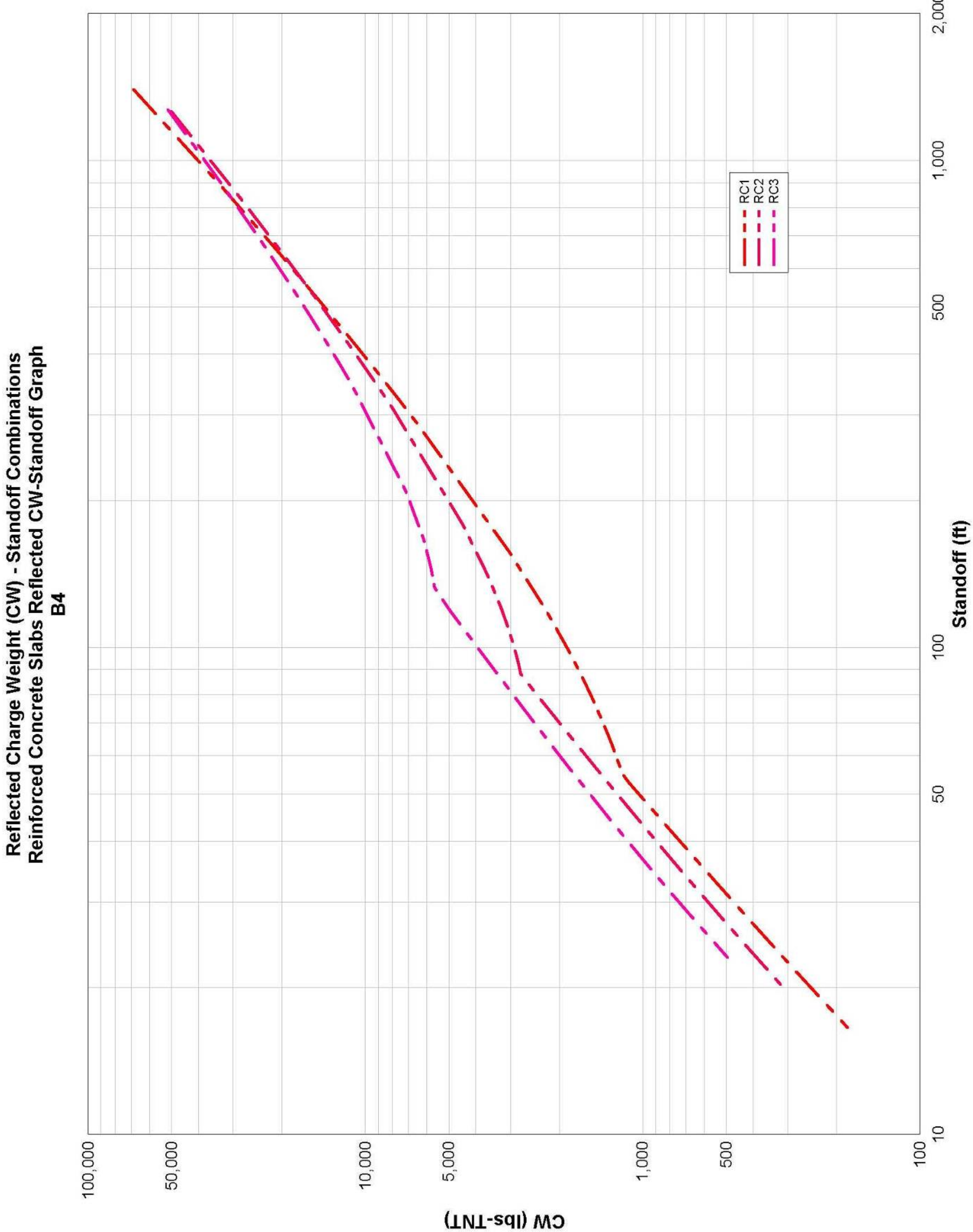
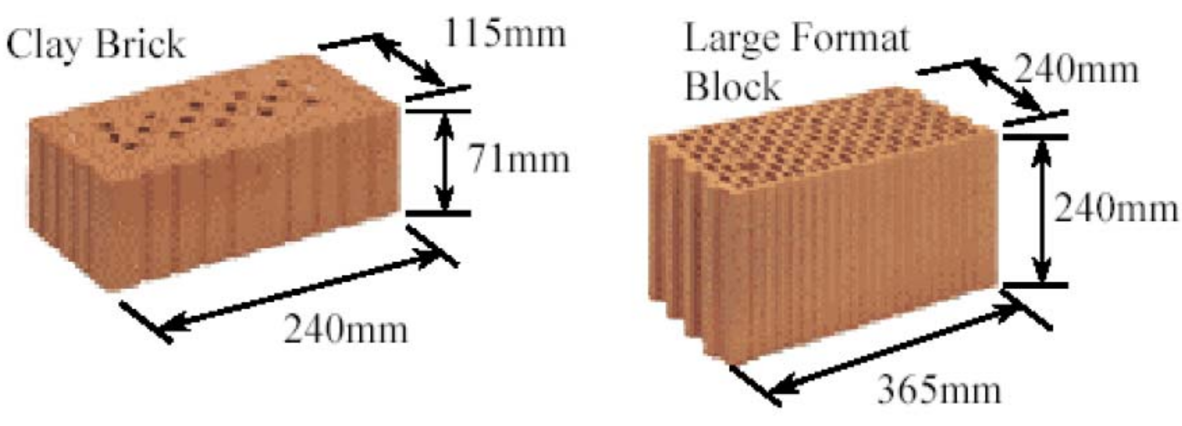


Figure E-3 – B4 Damage Curves for Reinforced Concrete

Appendix F - European Block Wall

European Block Analysis Summary:

1. Block Type:
    a. DIN: 105 Teil 1 + 2/HLz B
2. Reinforcement:
    a. Unreinforced
3. Walls:
    a. Thickness: 6 and 8 inch
    b. Heights: 10 and 12 feet
4. European Block Properties:
    a. Wall self-weight, 43.2 psf 6 inch, 57.6 psf 8 inch
    b. Masonry compression, f'_m: 1,800 psi
    c. Masonry, *DIF*: 1.19
5. Axial Load:
    a. 0 lb/inch
6. Support Conditions:
    a. Simple-Simple
    b. Brittle flexure
7. Tributary Widths:
    a. B_w. = 1
8. Wall Layups:
    a. EB1 – 10' tall, 6" thick with 10 psf support weight
    b. EB2 – 10' tall, 8" thick with 10 psf support weight
    c. EB3 – 12' tall, 6" thick with 10 psf support weight
    d. EB4 – 12' tall, 8" thick with 10 psf support weight



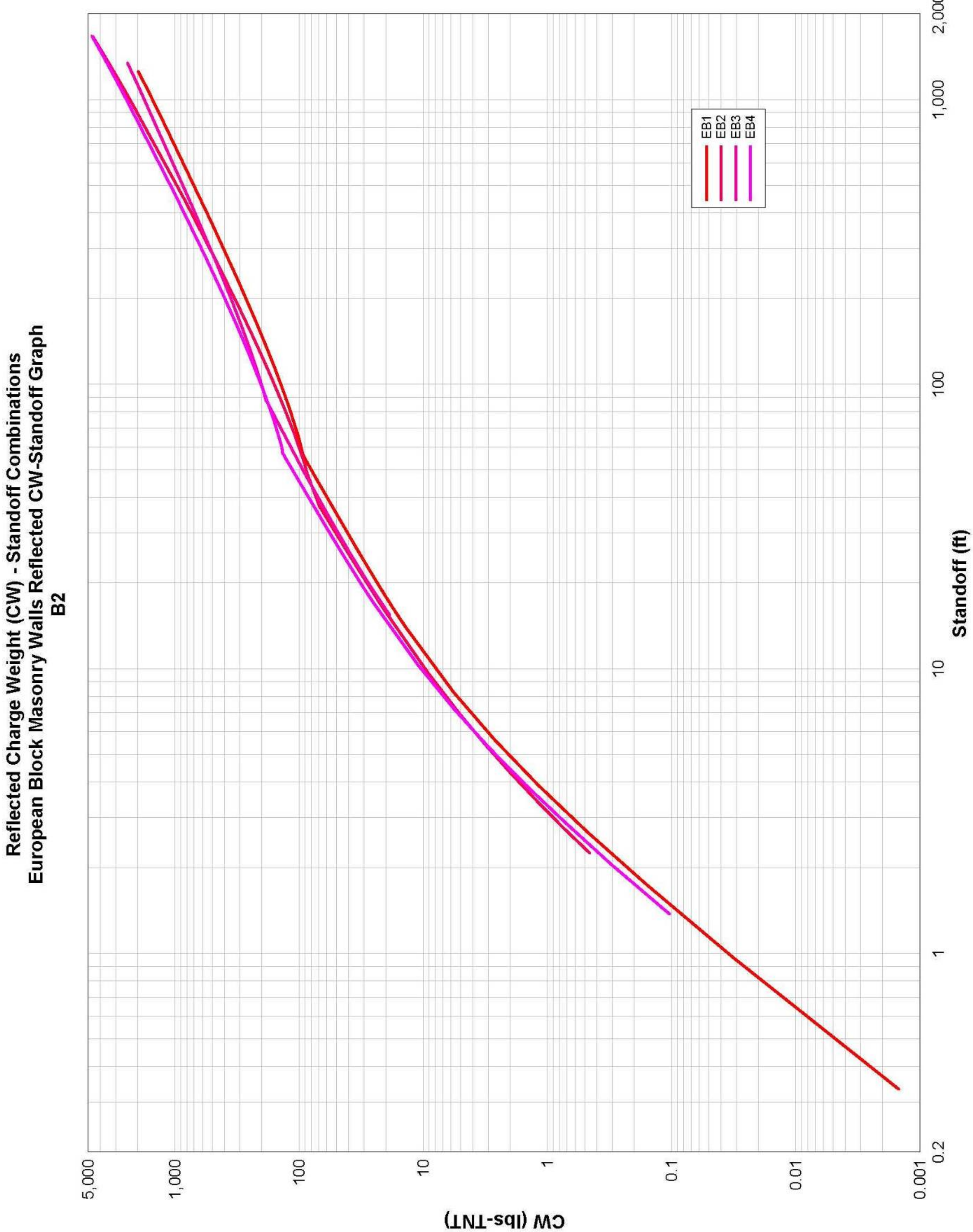


Figure F-1 – B2 Damage Curves for Unreinforced European Block

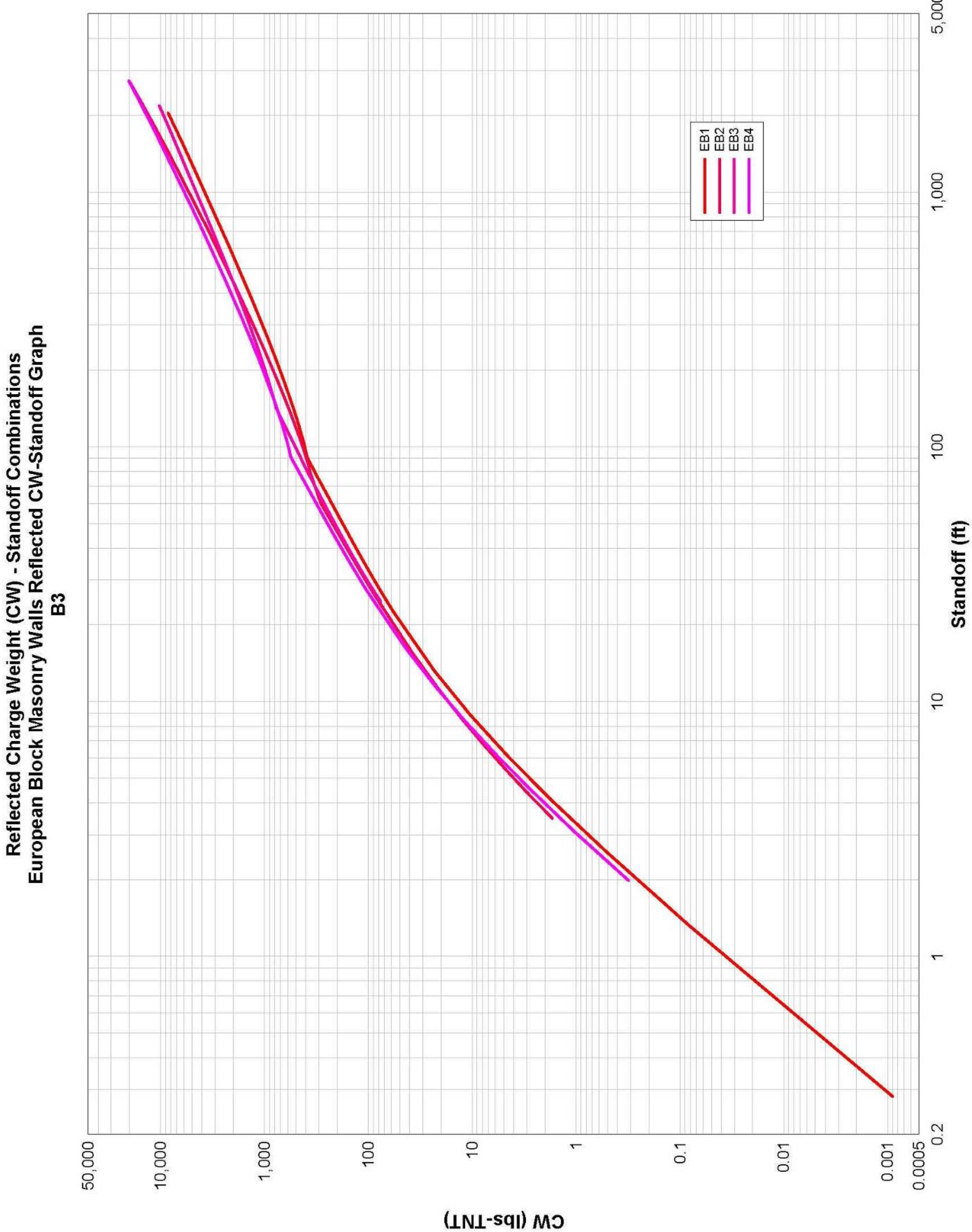


Figure F-2 – B3 Damage Curves for Unreinforced European Block

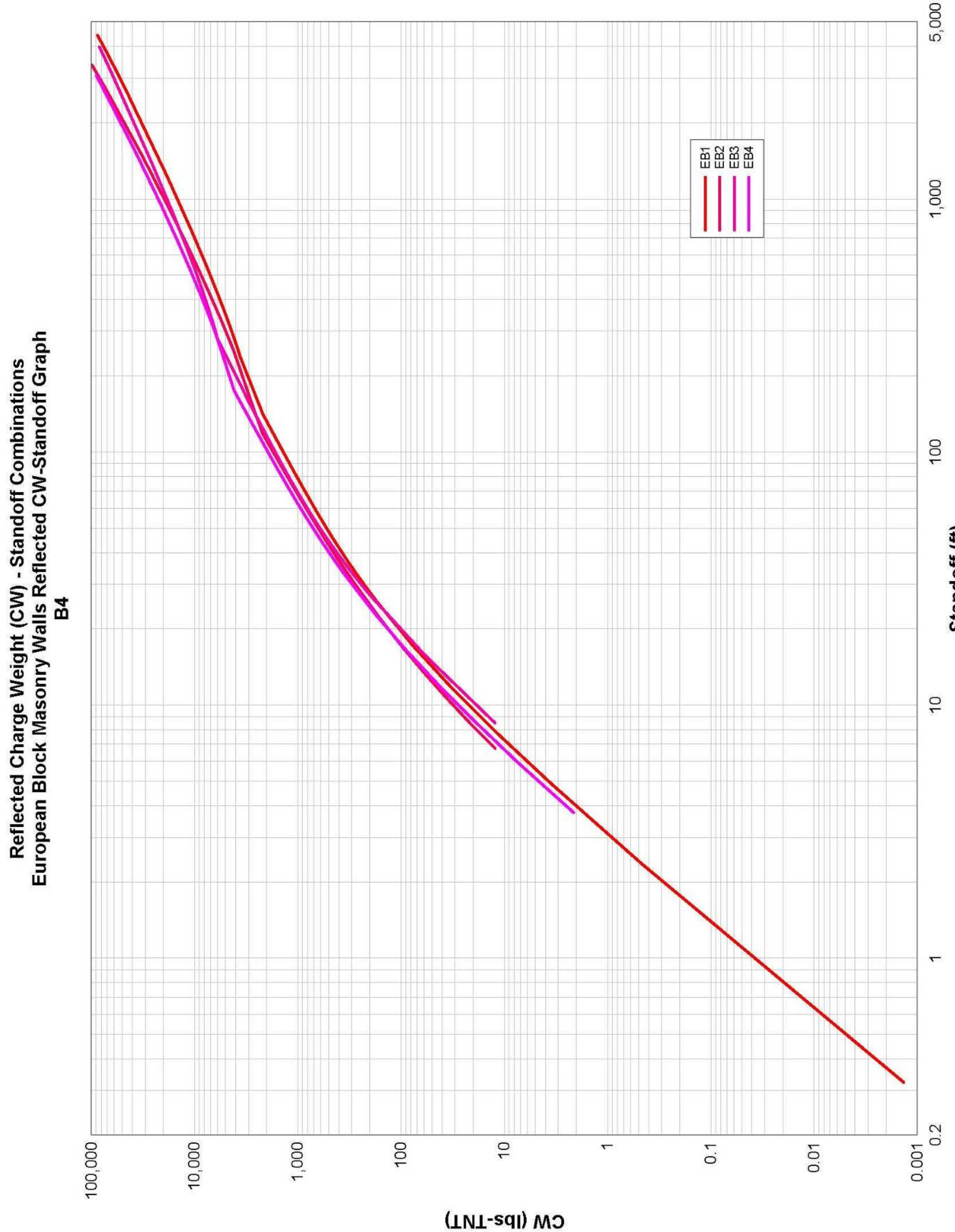


Figure F-3 – B4 Damage Curves for Unreinforced European Block

Appendix G - Metal Panel

Metal Panel Analysis Summary:

1. Sections:
    a. 1.5 and 3 inch deep section
    b. Gauges: 22, 20 and 18
2. Spans:
    a. 4, 6, and 8 feet
3. Shear Pullout Capacity
    a. V_c : Taken from Vulcraft catalog
4. Support Conditions:
    a. Simple-Simple
5. Panel Properties:
    a. Yield strength, F_y: 33, 000 psi
    b. Elastic modulus, E: 29,000,000 psi
    c. *SIF*: 1.21
    d. *DIF*: 1.1
    e. Dynamic yield strength, F_{dy}: 49,923 psi
6. Supported Weights:
    a. 10 psf
7. Wall Layups:
    a. MP1 – 4' span, 1.5" deep, 22 gauge
    b. MP2 – 4' span, 1.5" deep, 20 gauge
    c. MP3 – 4' span, 1.5" deep, 18 gauge
    d. MP4 – 4' span, 3" deep, 22 gauge
    e. MP5 – 4' span, 3" deep, 20 gauge
    f. MP6 – 4' span, 3" deep, 18 gauge
    g. MP7 – 6' span, 1.5" deep, 22 gauge
    h. MP8 – 6' span, 1.5" deep, 20 gauge
    i. MP9 – 6' span, 1.5" deep, 18 gauge
    j. MP10 – 6' span, 3" deep, 22 gauge
    k. MP11 – 6' span, 3" deep, 20 gauge
    l. MP12 – 6' span, 3" deep, 18 gauge
    m. MP13 – 8' span, 1.5" deep, 22 gauge
    n. MP14 – 8' span, 1.5" deep, 20 gauge
    o. MP15 – 8' span, 1.5" deep, 18 gauge
    p. MP16 – 8' span, 3" deep, 22 gauge
    q. MP17 – 8' span, 3" deep, 20 gauge
    r. MP18 – 8' span, 3" deep, 18 gauge

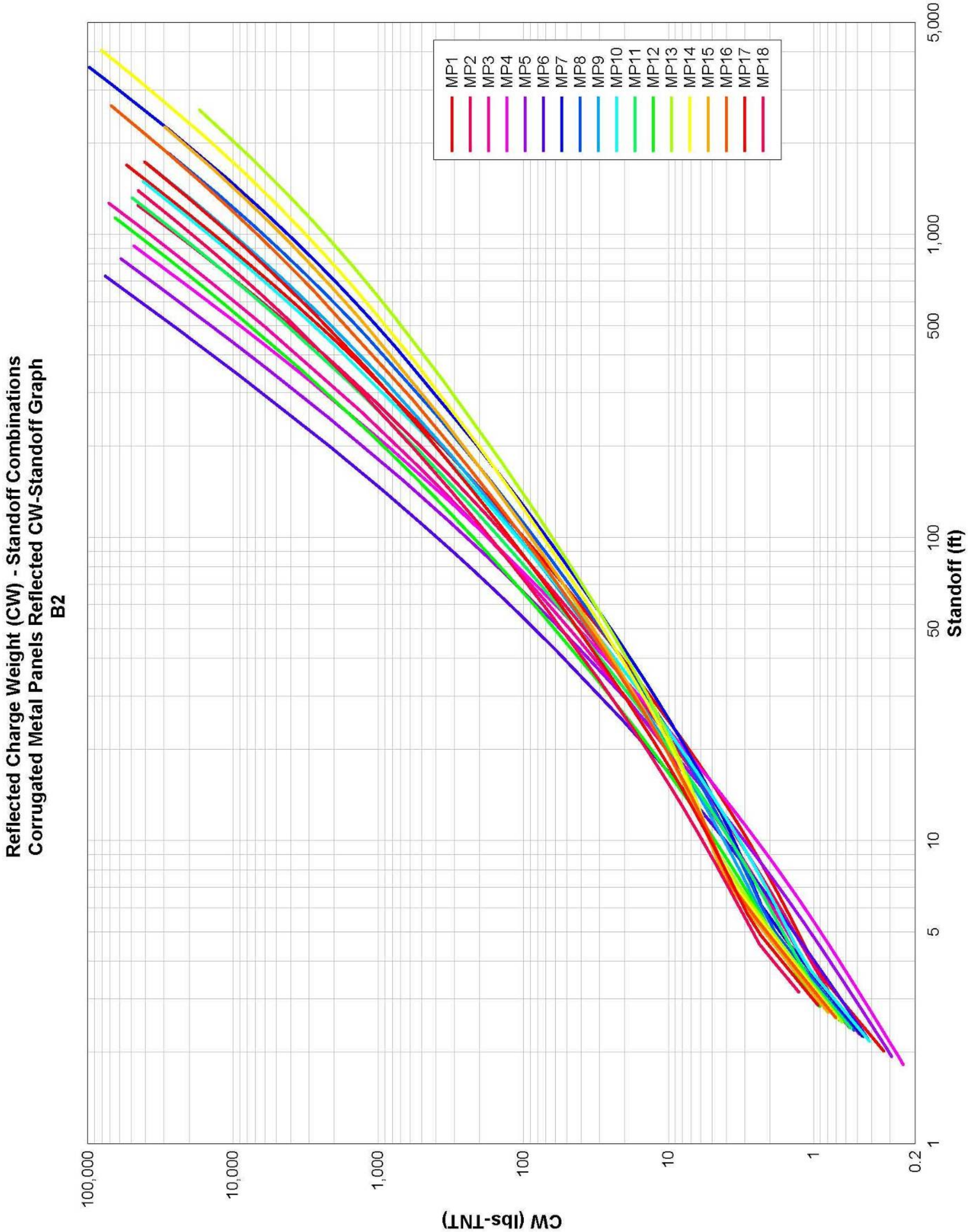


Figure G-1 – B2 Damage Curves for Metal Panels

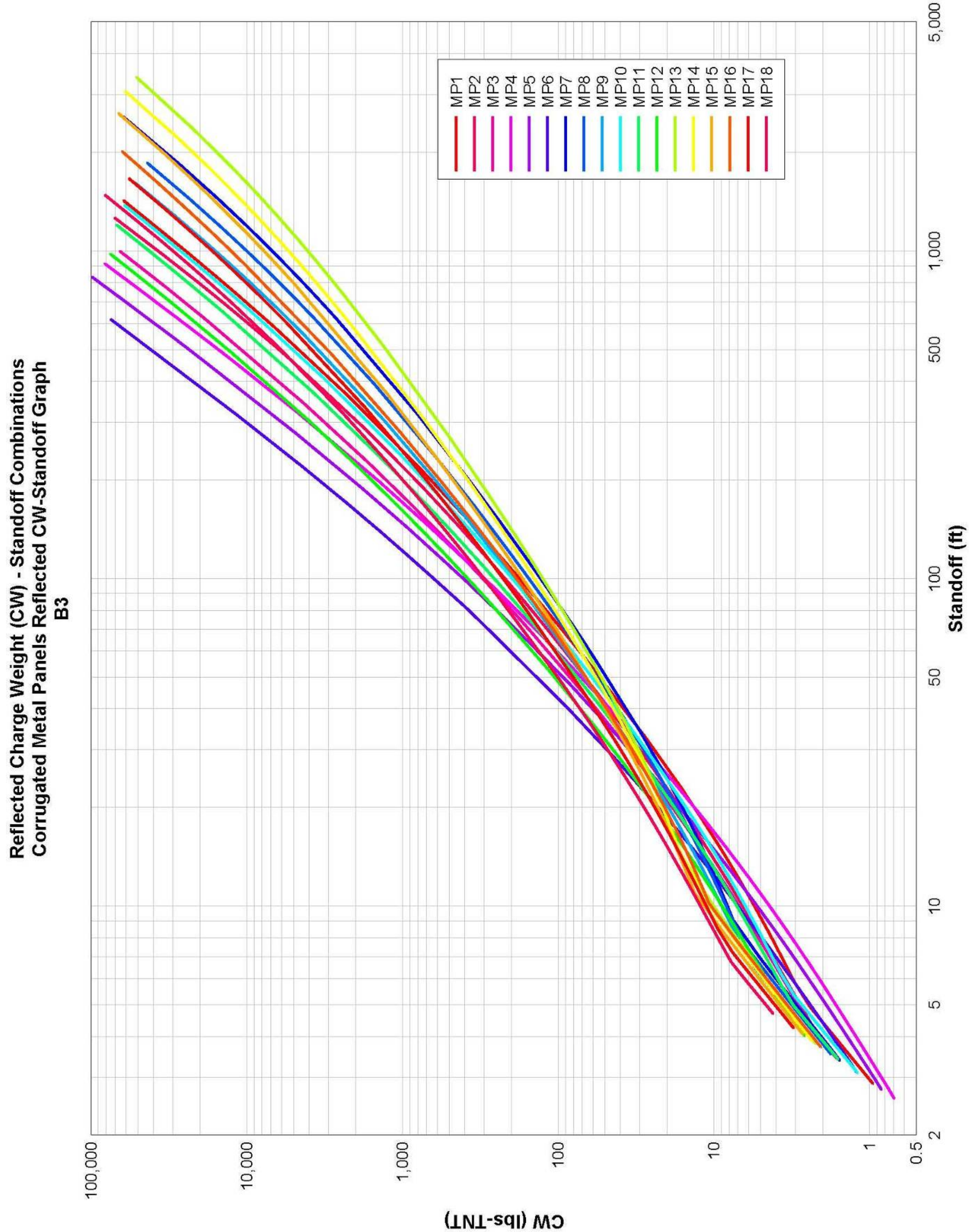


Figure G-2 – B3 Damage Curves for Metal Panels

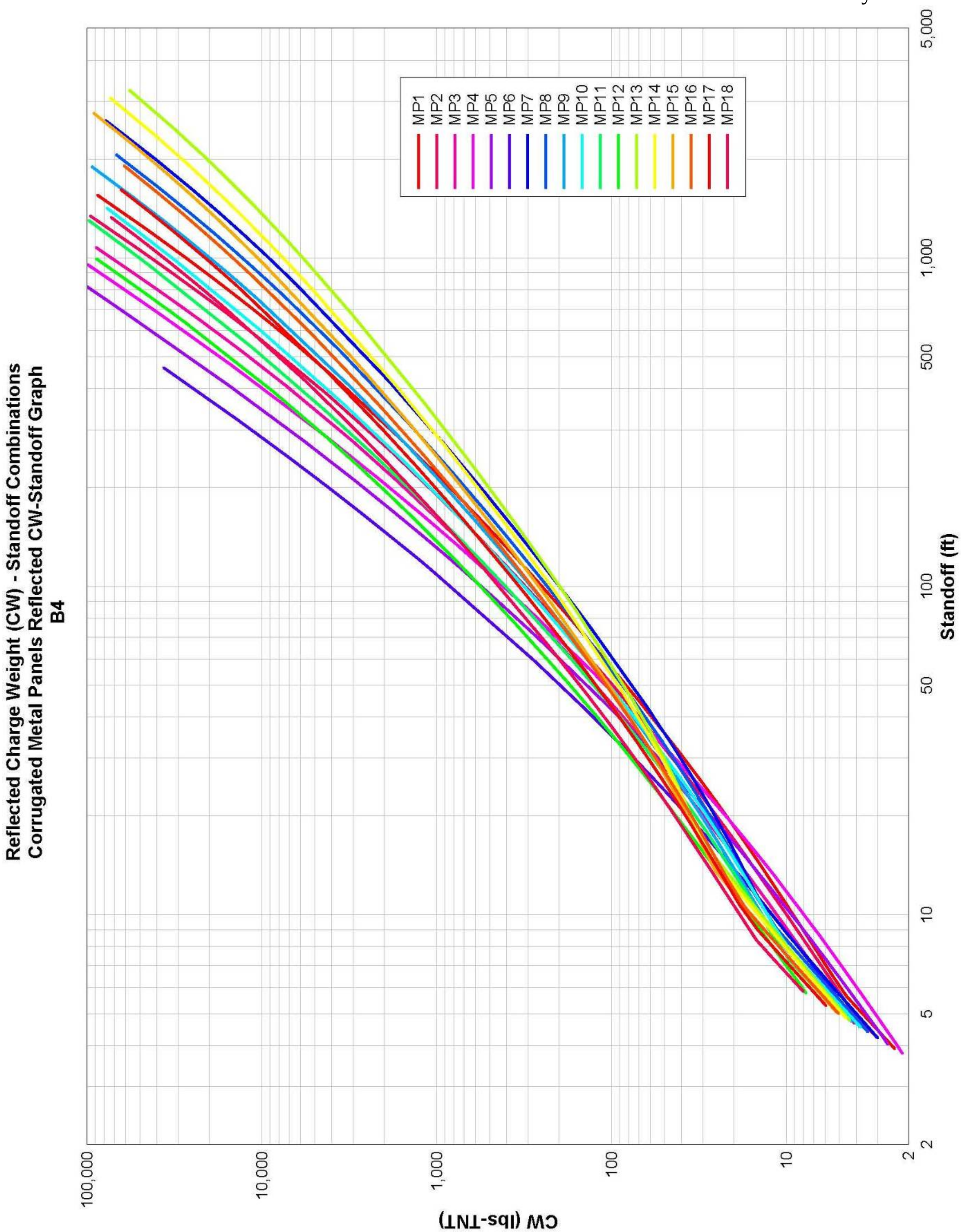


Figure G-3 – B4 Damage Curves for Metal Panels

Appendix H - Girts

Girt Analysis Summary:

1. Sections:
    a. 8Z3 and 10Z3
    b. Gauges: 16, 14 and 12
    c. 20 and 25 foot lengths
2. Girt Spacing's:
    a. 6 and 8 foot O.C.
3. Support Conditions:
    a. Simple-Simple
    b. Flexural
4. Stud Properties:
    a. Yield strength, F_y: 50, 000 psi, Grade 50
    b. Elastic modulus, E: 29,000,000 psi
    c. *SIF*: 1.05
    d. *DIF*: 1.19
    e. Dynamic yield strength, F_{dy}: 66,550 psi
5. Supported Weights:
    a. 5 psf
6. Wall Layups:
    a. G1- 20' span, 6' spacing, Z8x3 16 gauge
    b. G2- 20' span, 6' spacing, Z8x3 14 gauge
    c. G3- 20' span, 6' spacing, Z8x3 12 gauge
    d. G4- 20' span, 6' spacing, Z10x3 16 gauge
    e. G5- 20' span, 6' spacing, Z10x3 14 gauge
    f. G6- 20' span, 6' spacing, Z10x3 12 gauge
    g. G7- 20' span, 8' spacing, Z8x3 14 gauge
    h. G8- 20' span, 8' spacing, Z8x3 12 gauge
    i. G9- 25' span, 6' spacing, Z8x3 12 gauge
    j. G10- 25' span, 6' spacing, Z10x3 14 gauge
    k. G11- 25' span, 6' spacing, Z10x3 12 gauge
    l. G12- 25' span, 8' spacing, Z8x3 12 gauge
    m. G13- 25' span, 8' spacing, Z10x3 12 gauge

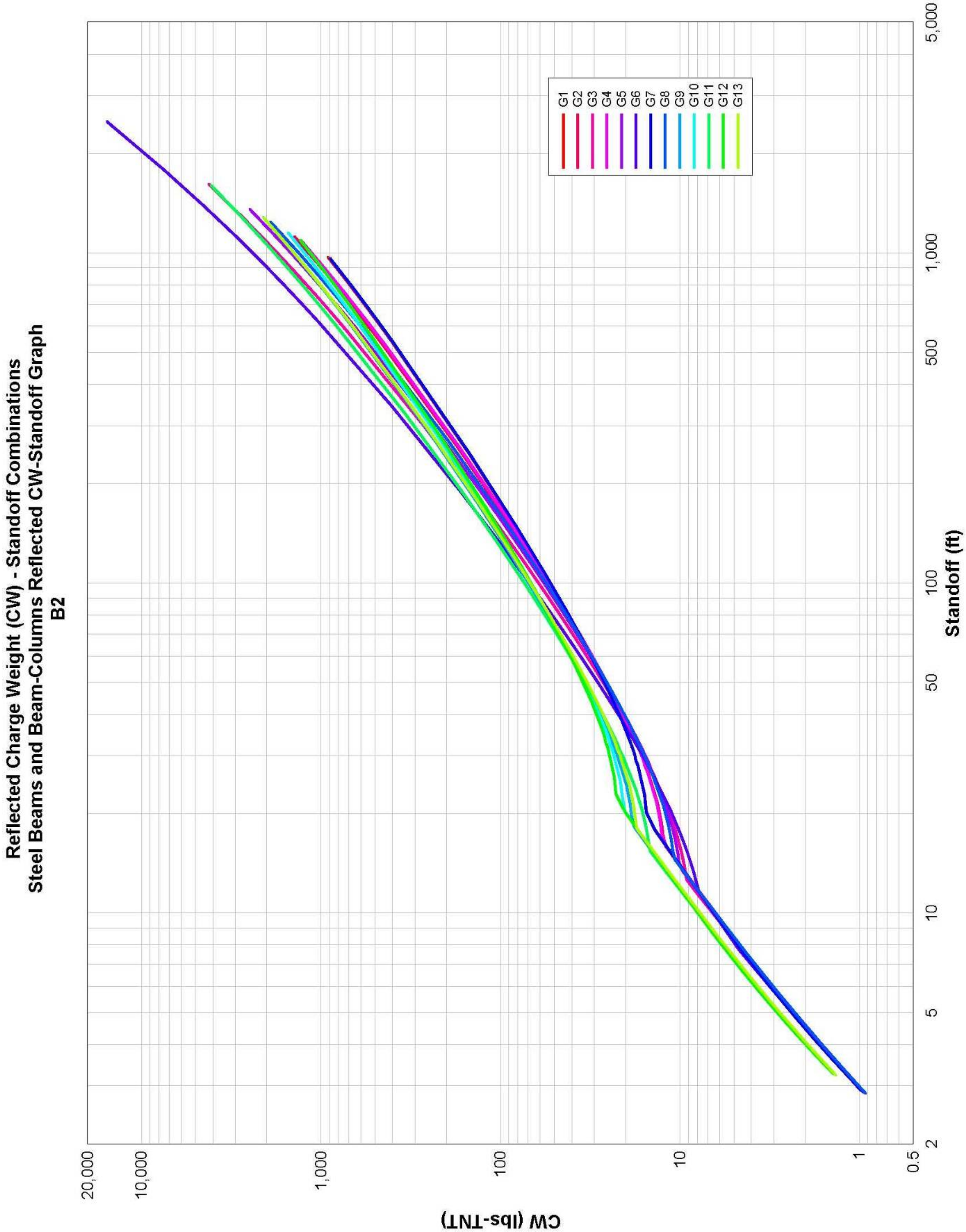


Figure H-1 – B2 Damage Curves for Girts

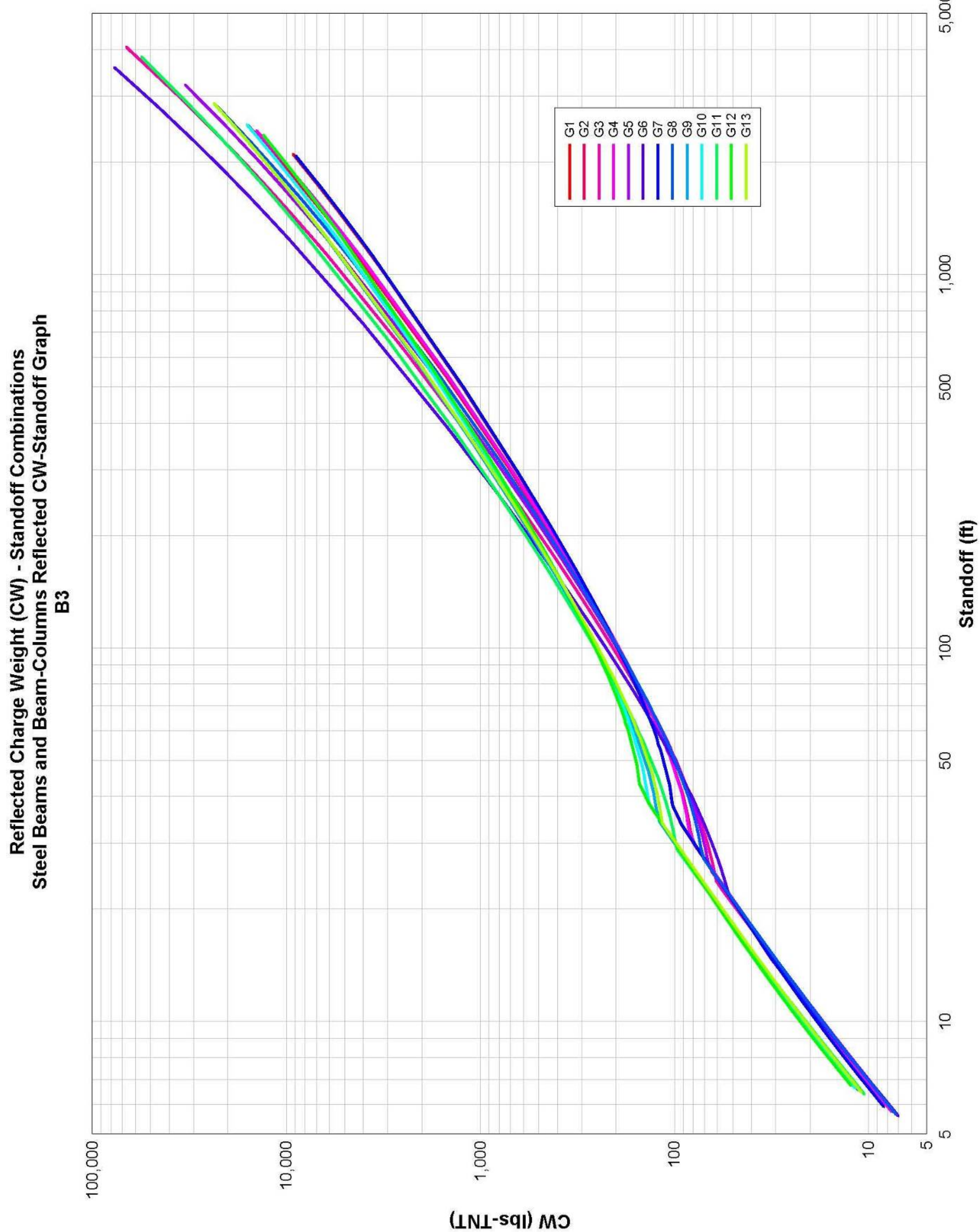


Figure H-2 – B3 Damage Curves for Girts

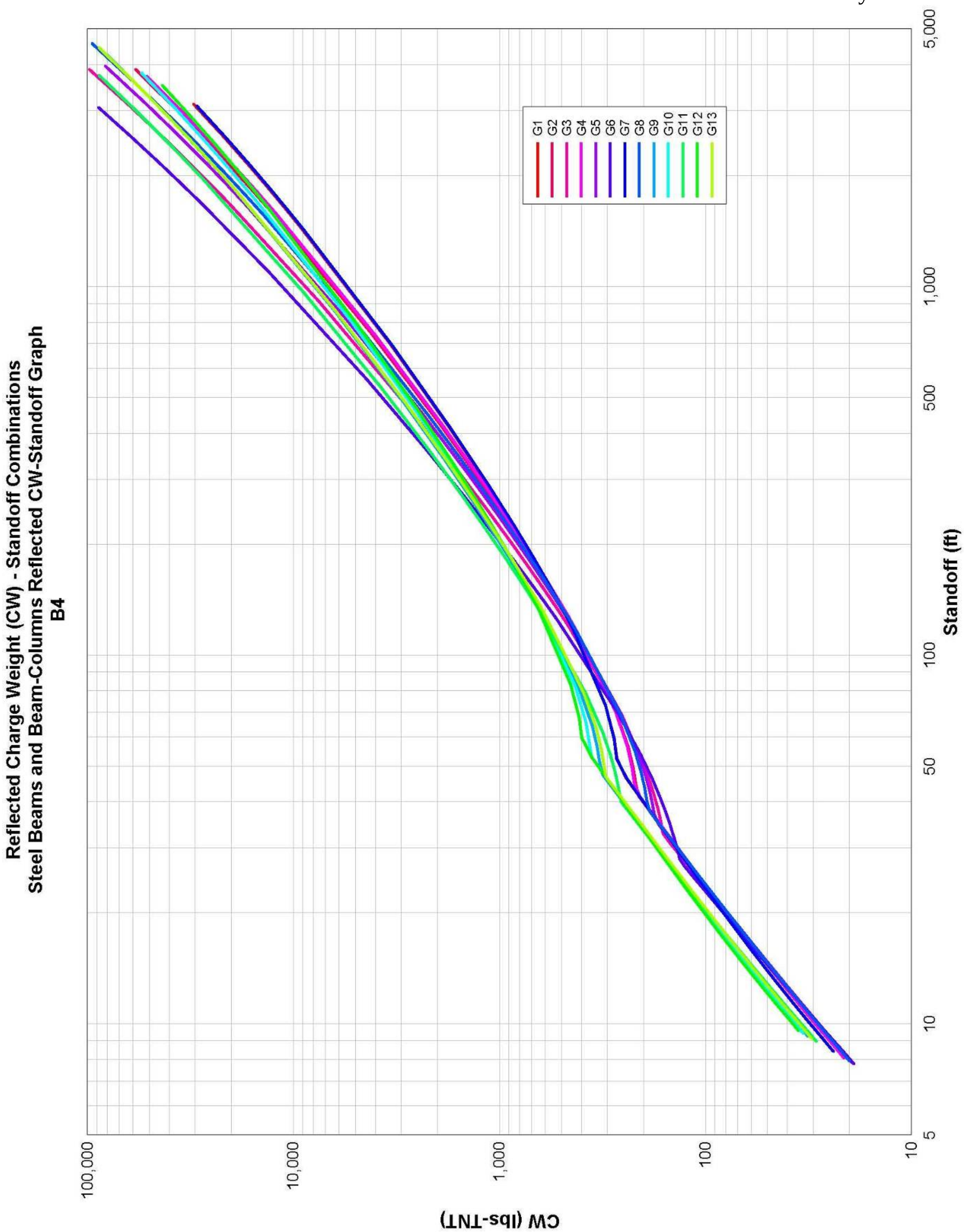


Figure H-3 – B4 Damage Curves for Girts

Appendix I - Windows

Window Standoff Summary:

1. Window Sizes
    a. Small: 24" x 48"
    b. Medium: 48" x 60"
    c. Large: 96" x 144"
2. Wingard Output
    a. No Hazard – HLOP
    a. Minimal Hazard – MLOP
    b. Very Low Hazard – LLOP
    c. Low Hazard – VLLOP
3. HazL Output
    a. No Hazard – HLOP
    b. Very Low Hazard – LLOP
    c. Low Hazard – VLLOP
4. Window Layups
    a. W1 – 24" x 48", 1/4" Laminated with 0.030" PVB
    b. W2 – 48" x 60", 1/4" Laminated with 0.030" PVB
    c. W3 – 96" x 144", 1/4" Laminated with 0.030" PVB
    d. W4 – 24" x 48", 1/4" Laminated with 0.060" PVB
    e. W5 – 48" x 60", 1/4" Laminated with 0.060" PVB
    f. W6 – 96" x 144", 1/4" Laminated with 0.060" PVB
    g. W7 – 24" x 48", 1" IGU - 1/4" AN monolithic outboard, 1/2" air gap, and 1/4" AN laminated with 0.030" PVB inboard
    h. W8 – 48" x 60", 1" IGU - 1/4" AN monolithic outboard, 1/2" air gap, and 1/4" AN laminated with 0.030" PVB inboard
    i. W9 – 96" x 144", 1" IGU - 1/4" AN monolithic outboard, 1/2" air gap, and 1/4" AN laminated with 0.030" PVB inboard

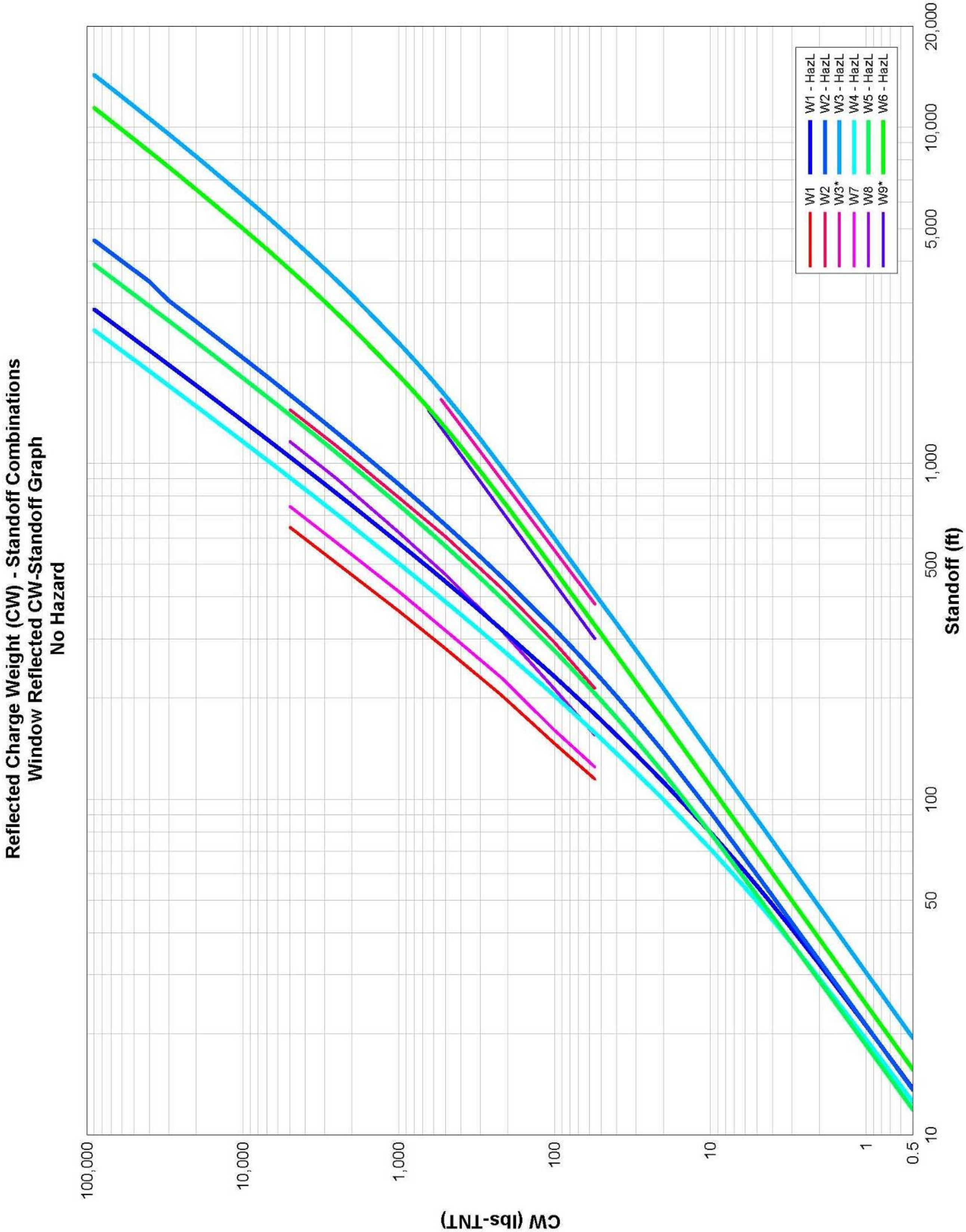


Figure I-1 – No Hazard Curves for Windows

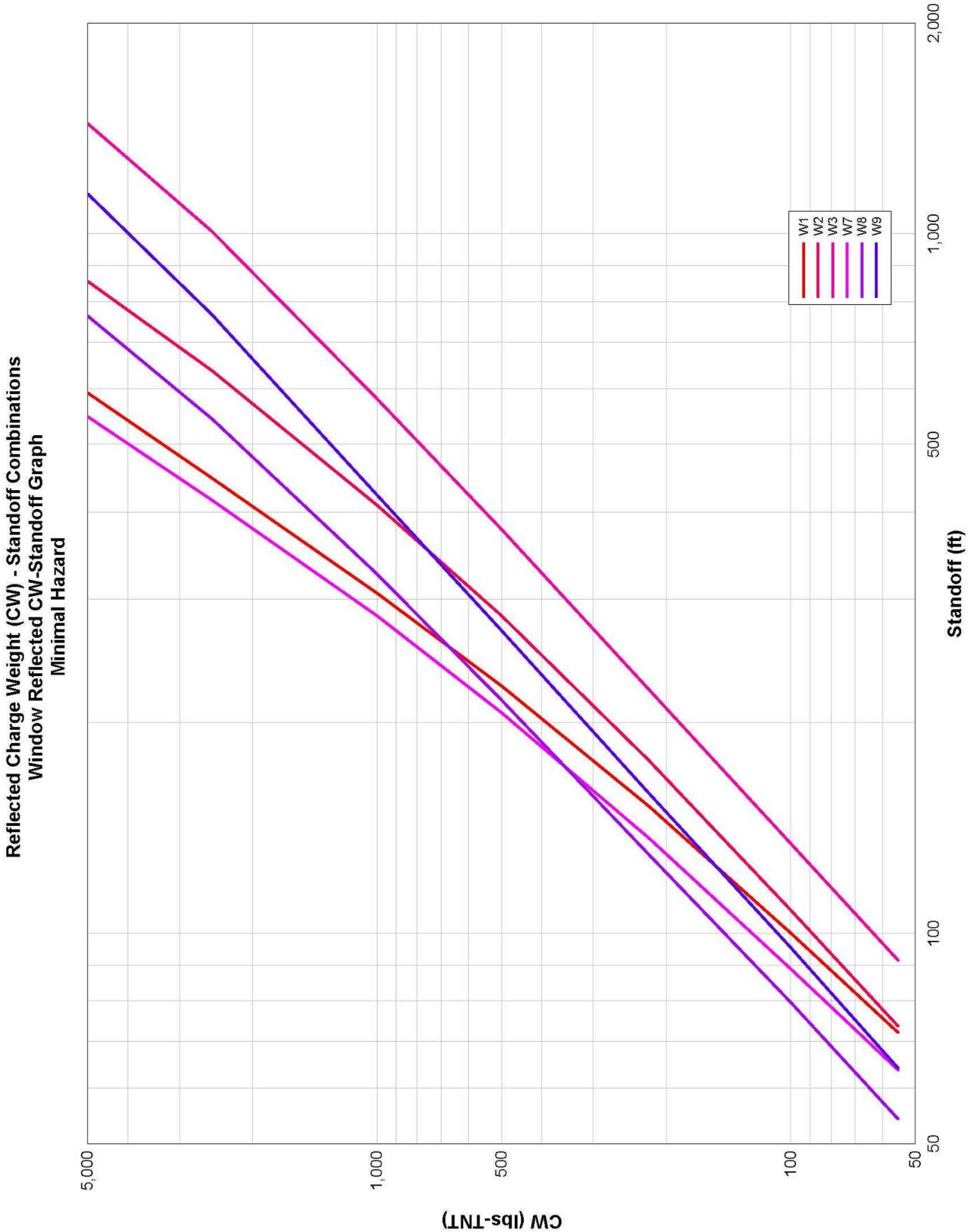


Figure I-2 – Minimal Hazard Curves for Windows

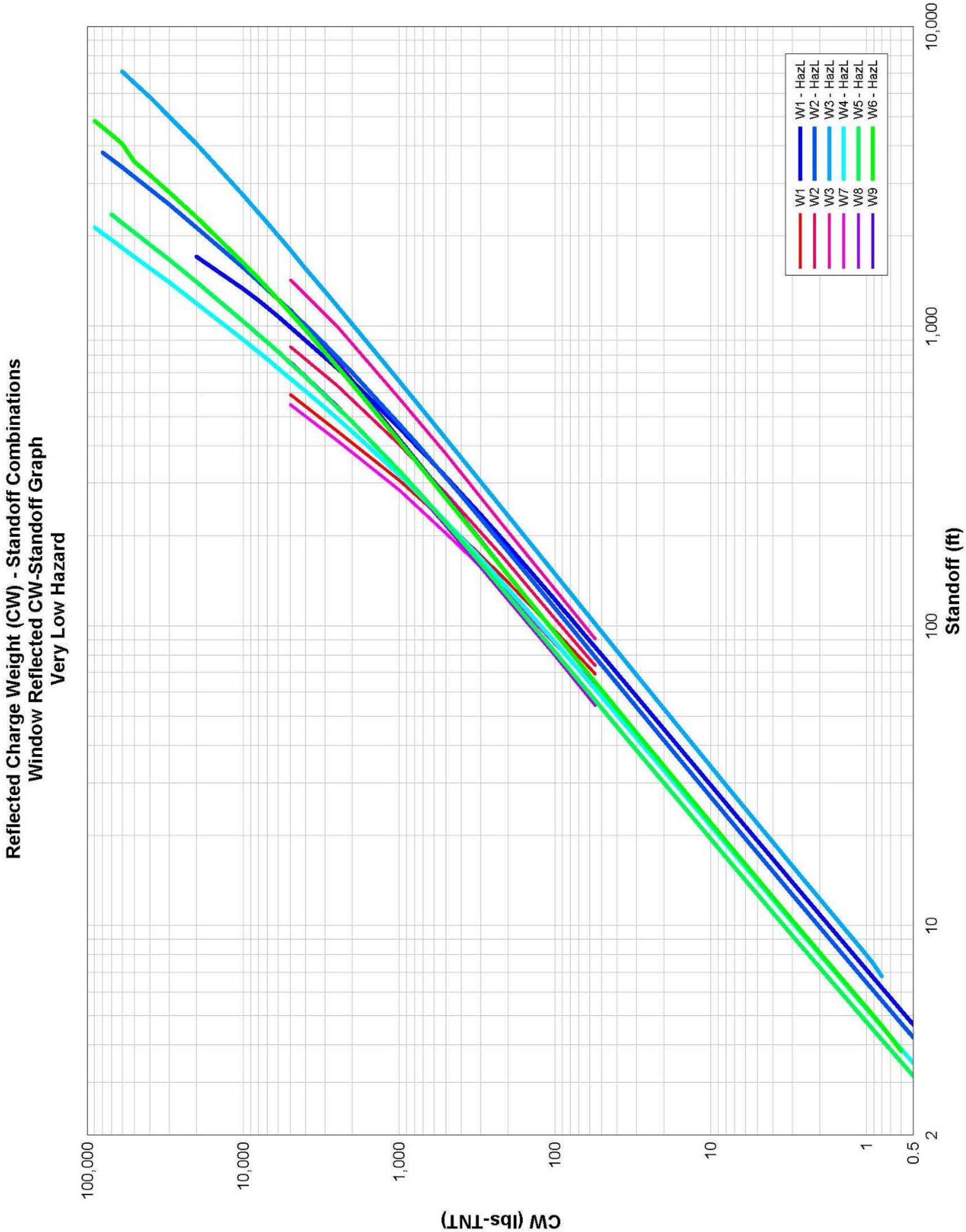


Figure I-3 – Very Low Hazard Curves for Windows

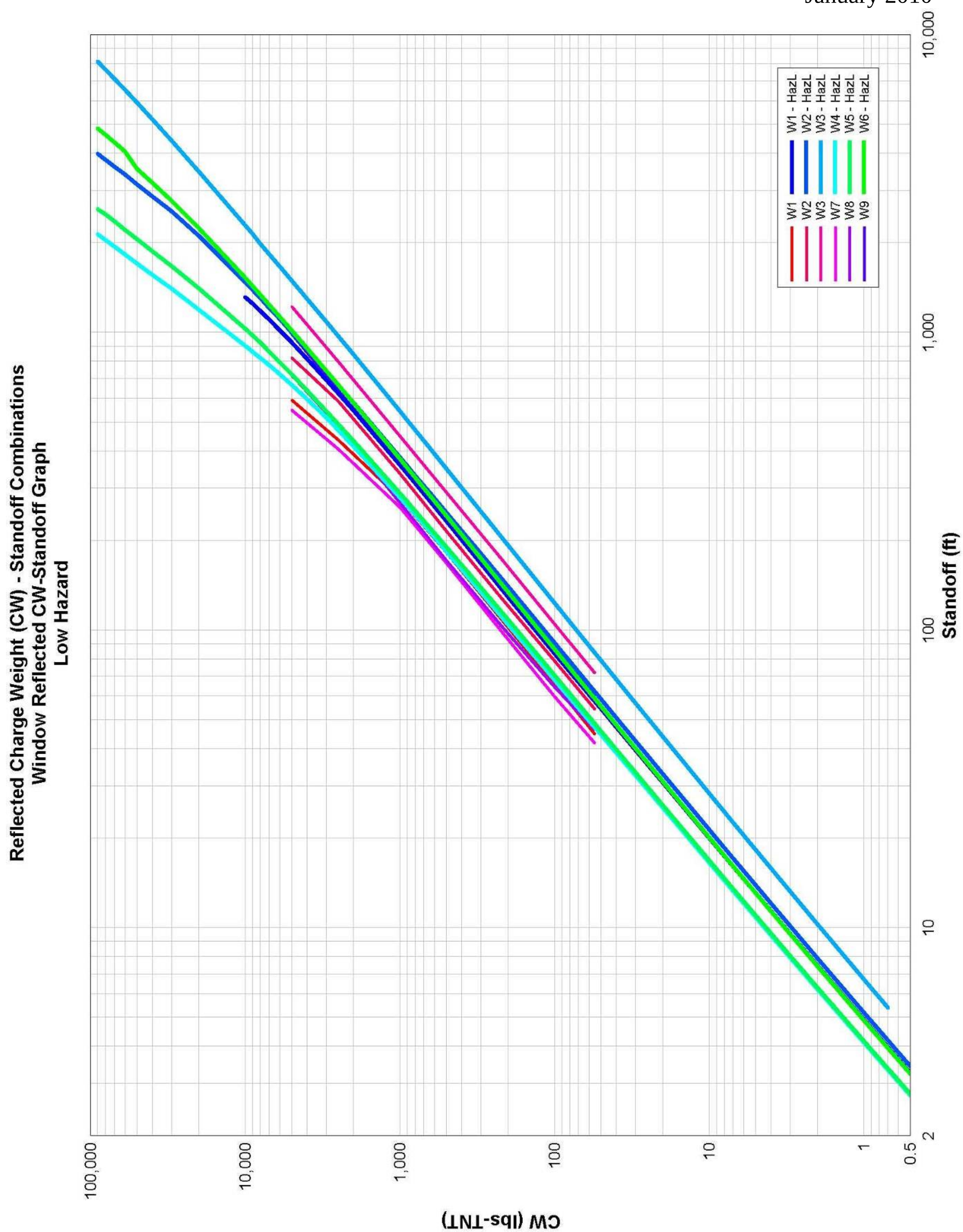


Figure I-4 – Low Hazard Curves for Windows

Appendix J - BICADS Data

BICADS (V2) Summary:

BICADS analysis matrix

1. Building:
    a. 240 feet wide, 48 feet deep, and 33 feet tall
    b. Roof pitch: 20 degrees
    c. Eve height: 9 and 10 feet
2. Charge location:
    a. Hemispherical: centered on 240 foot wall
    b. Charges: DoD I and DoD II
3. Standoff distances were based on lethality's to the occupants
    a. Occupants (66) were located within eight foot of the exterior walls
    b. Calculations located the regions within the building that saw "Serious Life-Threatening" injuries or worst. Standoff distances that created those injuries where used in this study.
    c. Windows: monolithic and annealed

Table J-1 – Lethality Based Standoff Distances for Light Framed, Light Clad Walls

Commercial Wood Frame Building with Wood Facing

Charge Weight	Span Height	Percent Glazing	6% Lethality	12% Lethality	18% Lethality
Charge Weight II	9’	0%	34’	-	-
		10%	34’	-	-
		20%	34’	-	-
		30%	34’	-	-
	10’	0%	32’	-	-
		10%	31’	-	-
		20%	31’	-	-
		30%	31’	-	-
Charge Weight I	9’	0%	90’	87‘	78’
		10%	89’	85‘	77’
		20%	88’	84‘	76’
		30%	83’	79‘	70’
	10’	0%	90’	86’	78’
		10%	89’	85’	76’
		20%	88’	84’	76’
		30%	82’	78’	68’

Commercial Metal Stud Building with Wood Facing

Charge Weight	Span Height	Percent Glazing	6% Lethality	12% Lethality	18% Lethality
Charge Weight II	9’	0%	34’	-	-
		10%	34’	-	-
		20%	34’	-	-
		30%	34’	-	-
	10’	0%	32’	-	-
		10%	31’	-	-
		20%	31’	-	-
		30%	31’	-	-
Charge Weight I	9’	0%	90’	87‘	78’
		10%	89’	85‘	77’
		20%	88’	84‘	76’
		30%	83’	79‘	70’
	10’	0%	90’	86’	78’
		10%	89’	85’	76’
		20%	88’	84’	76’
		30%	82’	78’	68’

Table J-2 – Lethality Based Standoff Distances for Light Framed, Heavily Clad Walls

Commercial Wood Frame Building with Brick Facing

Charge Weight	Span Height	Percent Glazing	1% Lethality	6% Lethality
Charge Weight II	9'	0%	-	-
		10%	9	-
		20%	9	-
		30%	9	-
	10'	0%	-	-
		10%	9	-
		20%	9	-
		30%	9	-
Charge Weight I	9'	0%	28	26
		10%	49	26
		20%	53	26
		30%	53	26
	10'	0%	23	21
		10%	49	21
		20%	53	21
		30%	53	21

Commercial Metal Stud Building with Brick Facing

Charge Weight	Span Height	Percent Glazing	1% Lethality	6% Lethality
Charge Weight II	9'	0%	-	-
		10%	9	-
		20%	9	-
		30%	9	-
	10'	0%	-	-
		10%	9	-
		20%	9	-
		30%	9	-
Charge Weight I	9'	0%	28	26
		10%	49	26
		20%	53	26
		30%	53	26
	10'	0%	23	21
		10%	49	21
		20%	53	21
		30%	53	21

Table J-3 – Lethality Based Standoff Distances for Reinforced Masonry

Reinforced Masonry Load-Bearing Building

Charge Weight	Span Height	Percent Glazing	1% Lethality
Charge Weight II	10	0%	-
		10%	9
		20%	9
		30%	9
	12	0%	-
		10%	9
		20%	9
		30%	9
	14	0%	-
		10%	9
		20%	9
		30%	9
Charge Weight I	10	0%	19
		10%	49
		20%	53
		30%	53
	12	0%	17
		10%	49
		20%	53
		30%	53
	14	0%	16
		10%	49
		20%	53
		30%	53

Building

Building Name:	Wood Barracks 10'	
Building Type:	12.Commercial Wood Frame Building	
Detailed Input:	No	
Building Height:	33.00 (ft)	
Number of Floors:	3	
Roof Pitch:	20.00 (deg)	
Building Width:	48.00 (ft)	
Building Length:	240.00 (ft)	
Xc:	120.00 (ft)	
Yc:	24.00 (ft)	
Orientation:	0.00 (deg)	
Building Coordinates:	X (ft)	Y (ft)
	0.00	48.00
	240.00	48.00
	240.00	0.00
	0.00	0.00
Total Occupancy:	66	
Occupancy Type:	Simplified	
% Interior Population:	0	
% Perimeter Population:	100	

Blast Source

Blast Name:	DoD I
Blast Type:	Charge Weight
Charge Weight:	Charge Weight II
Charge Type:	TNT
Charge Coordinates (ft)	X: 120.00
	Y: -90.00
	Z: 0.00

Minimum Charge Standoff Distance to Building Component:	90.48 (ft)
Minimum Scaled Standoff Distance to Building Component:	15.01 (ft)

Components

Location	Name	Type	Property	Value
North Wall	North Wall Comp	Metal/Wood Stud Wall		
			Span Length	10 ft
			Stud Type	2" x 6" Wood Stud at 16 inch
			Exterior Wall Cladding	Wood/Metal Panel
East Wall	East Wall Comp	Metal/Wood Stud Wall		
			Span Length	10 ft
			Stud Type	2" x 6" Wood Stud at 16 inch
			Exterior Wall Cladding	Wood/Metal Panel
South Wall	South Wall Comp	Metal/Wood Stud Wall		
			Span Length	10 ft
			Stud Type	2" x 6" Wood Stud at 16 inch
			Exterior Wall Cladding	Wood/Metal Panel
West Wall	West Wall Comp	Metal/Wood Stud Wall		
			Span Length	10 ft
			Stud Type	2" x 6" Wood Stud at 16 inch
			Exterior Wall Cladding	Wood/Metal Panel
Roof	Roof Comp	Wood Roof System		
			Span Length	20 ft
			Stud Type	2" x 6" Wood Stud at 16 inch
			Roof Cladding	Built-up Roof Over Plywood Deck
Floor	Floor Comp	Office/Residential Floor System		
Interior		Residential/Dormitory Interior		

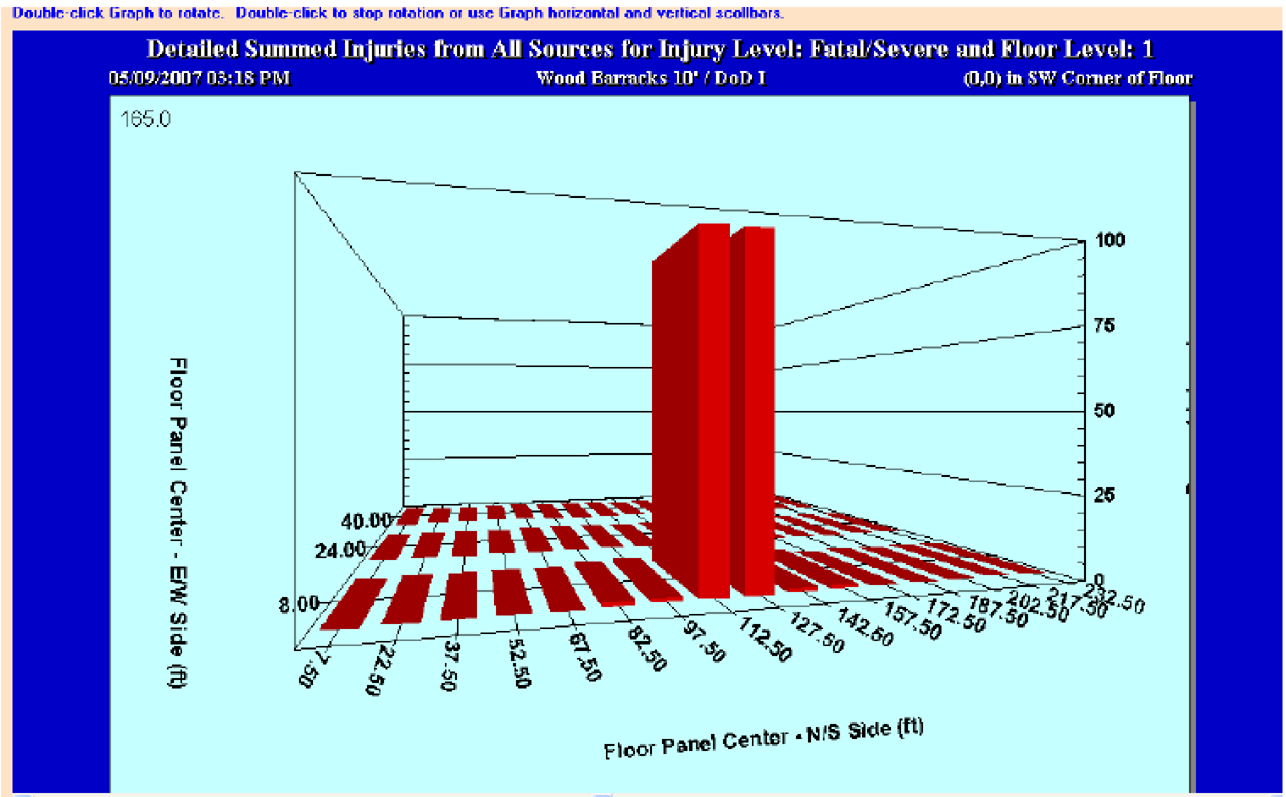
Calculated Blast Loads Summary

	Maximum Load Based on Highest Pressure				Minimum Load Based on Lowest Pressure			
Side	Peak Pressure (psi)	Impulse (psi-ms)	Net* Impulse (psi-ms)	Floor	Peak Pressure (psi)	Impulse (psi-ms)	Net* Impulse (psi-ms)	Floor
North Wall	0.81	18.83	18.83	1	0.81	18.83	18.83	3
East Wall	2.04	20.11	20.11	1	1.57	16.62	16.62	3
South Wall	10.23	69.86	69.86	1	4.77	34.28	34.28	3
West Wall	2.04	20.11	20.11	1	1.57	16.62	16.62	3
Roof	3.74	29.94	29.94	0	1.76	18.09	18.09	0
Floor	0.00	0.00	0.00	0	0.00	0.00	0.00	3

***Net impulse, equal to applied exterior impulse minus room fill pressure impulse, used to get damage from wall and roof components causing injuries. See report for more information on net impulse calculation assumptions.**

Total Building Injury Summary**

Area	Percentage of Injured Occupants					Number of Injured Occupants				
	% Fatal/Severe Injury	% Serious Life-Threatening Injury	% Serious Non-Life Threatening Injury	% Minor to Moderate Injury	No Calculated Injury	Fatal/Severe Injury	Serious Life-Threatening Injury	Serious Non-Life Threatening Injury	Minor to Moderate Injury	No Calculated Injury
Total Building	6	0	0	1	93	4	0	0	0	62
Floor 1	6	0	0	1	93	1	0	0	0	20.78
Floor 2	6	0	0	1	93	1	0	0	0	20.78
Floor 3	6	0	0	0	94	1	0	0	0	20.78



Building

Building Name:	Wood Barracks 10'	
Building Type:	13.Commercial Metal Stud Building	
Detailed Input:	No	
Building Height:	33.00 (ft)	
Number of Floors:	3	
Roof Pitch:	20.00 (deg)	
Building Width:	48.00 (ft)	
Building Length:	240.00 (ft)	
Xc:	120.00 (ft)	
Yc:	24.00 (ft)	
Orientation:	0.00 (deg)	
Building Coordinates:	X (ft)	Y (ft)
	0.00	48.00
	240.00	48.00
	240.00	0.00
	0.00	0.00
Total Occupancy:	66	
Occupancy Type:	Simplified	
% Interior Population:	0	
% Perimeter Population:	100	

Blast Source

Blast Name:	DoD I
Blast Type:	Charge Weight
Charge Weight:	Charge Weight II
Charge Type:	TNT
Charge Coordinates (ft)	X: 120.00
	Y: -90.00
	Z: 0.00

Minimum Charge Standoff Distance to Building Component: 90.48 (ft)
Minimum Scaled Standoff Distance to Building Component: 15.01 (ft)

Components

Location	Name	Type	Property	Value
North Wall	North Wall Comp	Metal/Wood Stud Wall		
			Span Length	10 ft
			Stud Type	6" Steel Stud at 16 inch
			Exterior Wall Cladding	Wood/Metal Panel
East Wall	East Wall Comp	Metal/Wood Stud Wall		
			Span Length	10 ft
			Stud Type	6" Steel Stud at 16 inch
			Exterior Wall Cladding	Wood/Metal Panel
South Wall	South Wall Comp	Metal/Wood Stud Wall		
			Span Length	10 ft
			Stud Type	6" Steel Stud at 16 inch
			Exterior Wall Cladding	Wood/Metal Panel
West Wall	West Wall Comp	Metal/Wood Stud Wall		
			Span Length	10 ft
			Stud Type	6" Steel Stud at 16 inch
			Exterior Wall Cladding	Wood/Metal Panel
Roof	Roof Comp	Open Web Steel Joists		
			Span Length	20 ft
			Allowable Load Capacity	50 psf
			Load Causing L/360 Deflection	30 psf
			Joist Weight	1.25 psf
			Roof Panel Weight	3 psf
Floor	Floor Comp	Office/Residential Floor System		
Interior		Residential/Dormitory Interior		

Calculated Blast Loads Summary

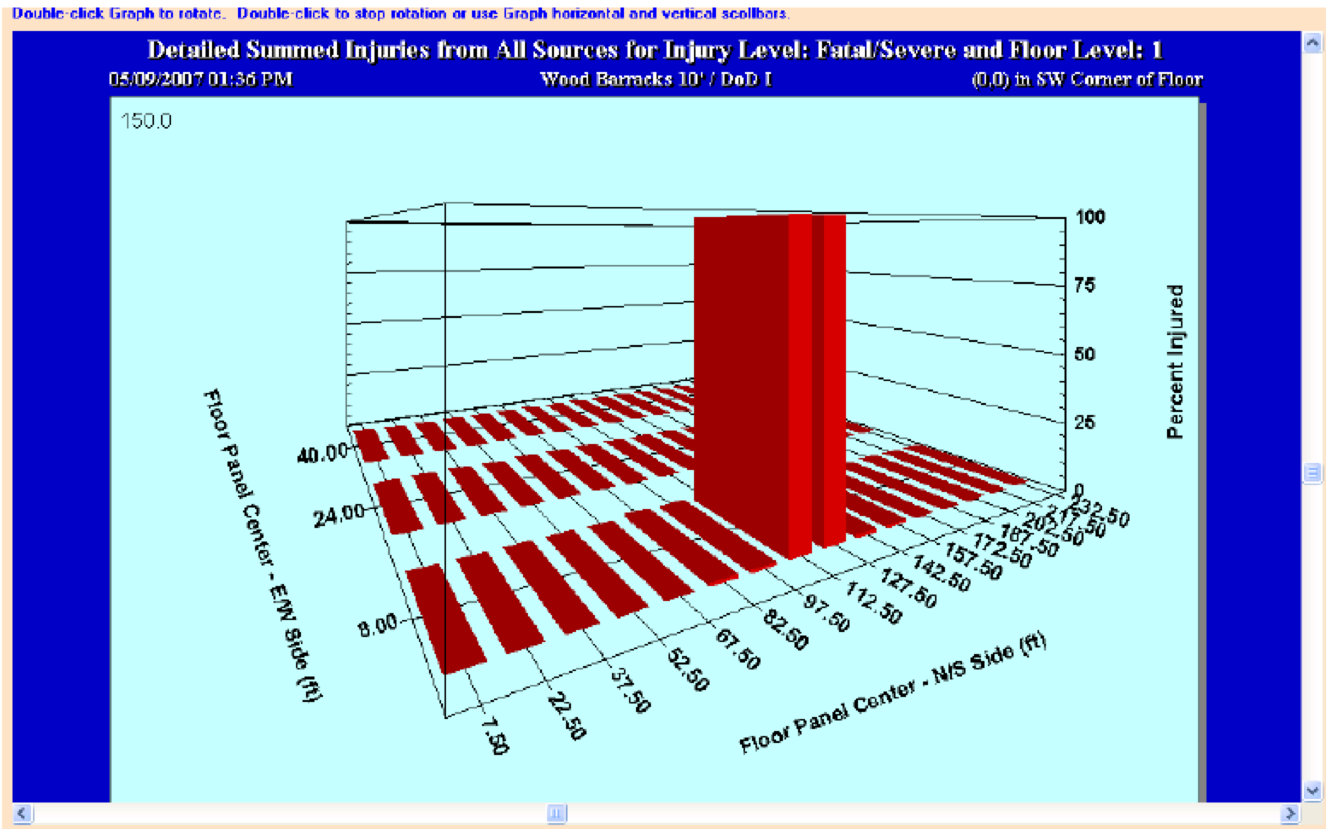
	Maximum Load Based on Highest Pressure				Minimum Load Based on Lowest Pressure			
Side	Peak Pressure (psi)	Impulse (psi-ms)	Net* Impulse (psi-ms)	Floor	Peak Pressure (psi)	Impulse (psi-ms)	Net* Impulse (psi-ms)	Floor
North Wall	0.81	18.83	18.83	1	0.81	18.83	18.83	3
East Wall	2.04	20.11	20.11	1	1.57	16.62	16.62	3
South Wall	10.23	69.86	69.86	1	4.77	34.28	34.28	3
West Wall	2.04	20.11	20.11	1	1.57	16.62	16.62	3
Roof	3.74	29.94	29.94	0	1.76	18.09	18.09	0
Floor	0.00	0.00	0.00	0	0.00	0.00	0.00	3

*Net impulse, equal to applied exterior impulse minus room fill pressure impulse, used to get damage from wall and roof components causing injuries. See report for more information on net impulse calculation assumptions.

Total Building Injury Summary**

	Percentage of Injured Occupants					Number of Injured Occupants				
Area	% Fatal/Severe Injury	% Serious Life-Threatening Injury	% Serious Non-Life Threatening Injury	% Minor to Moderate Injury	No Calculated Injury	Fatal/Severe Injury	Serious Life-Threatening Injury	Serious Non-Life Threatening Injury	Minor to Moderate Injury	No Calculated Injury
Total Building	6	0	0	1	93	4	0	0	0	62
Floor 1	6	0	0	1	93	1	0	0	0	20.78
Floor 2	6	0	0	1	93	1	0	0	0	20.78
Floor 3	6	0	0	0	94	1	0	0	0	20.78

**Use Injury Level Description Button on Bottom of Reports Form to Print/Copy to Clipboard Injury Level Descriptions



Building

Building Name:	Masonry Barracks 10'	
Building Type:	14.Reinforced Masonry Load-Bearing Building	
Detailed Input:	No	
Building Height:	33.00 (ft)	
Number of Floors:	3	
Roof Pitch:	20.00 (deg)	
Building Width:	48.00 (ft)	
Building Length:	240.00 (ft)	
Xc:	120.00 (ft)	
Yc:	24.00 (ft)	
Orientation:	0.00 (deg)	
Building Coordinates:	X (ft)	Y (ft)
	0.00	48.00
	240.00	48.00
	240.00	0.00
	0.00	0.00
Total Occupancy:	66	
Occupancy Type:	Simplified	
% Interior Population:	0	
% Perimeter Population:	100	

Blast Source

Blast Name:	DoD I
Blast Type:	Charge Weight
Charge Weight:	Charge Weight II
Charge Type:	TNT
Charge Coordinates (ft)	X: 120.00
	Y: -20.00
	Z: 0.00

Minimum Charge Standoff Distance to Building Component: 22.06 (ft)
Minimum Scaled Standoff Distance to Building Component: 3.66 (ft)

Components

Location	Name	Type	Property	Value
North Wall	North Wall Comp	One-Way Reinforced Masonry Wall		
			Span Length	10 ft
			Wall Thickness	7.63 inch
			Masonry Type	CMU
			Reinforcing	Medium (#4 @ 16 inch)
			Reinforcement Location	Center of Wall
			Additional Non-Structural Wall Weight	0 psf
			Boundary Conditions	Simple Supports
East Wall	East Wall Comp	One-Way Reinforced Masonry Wall		
			Span Length	10 ft
			Wall Thickness	7.63 inch
			Masonry Type	CMU
			Reinforcing	Medium (#4 @ 16 inch)
			Reinforcement Location	Center of Wall
			Additional Non-Structural Wall Weight	0 psf
			Boundary Conditions	Simple Supports
South Wall	South Wall Comp	One-Way Reinforced Masonry Wall		
			Span Length	10 ft
			Wall Thickness	7.63 inch
			Masonry Type	CMU
			Reinforcing	Medium (#4 @ 16 inch)
			Reinforcement Location	Center of Wall
			Additional Non-Structural Wall Weight	0 psf
			Boundary Conditions	Simple Supports
West Wall	West Wall Comp	One-Way Reinforced Masonry Wall		
			Span Length	10 ft
			Wall Thickness	7.63 inch
			Masonry Type	CMU
			Reinforcing	Medium (#4 @ 16 inch)
			Reinforcement Location	Center of Wall

Location	Name	Type	Property	Value
			Additional Non-Structural Wall Weight	0 psf
			Boundary Conditions	Simple Supports
Roof	Roof Comp	Open Web Steel Joists		
			Span Length	20 ft
			Allowable Load Capacity	50 psf
			Load Causing L/360 Deflection	30 psf
			Joist Weight	1.25 psf
			Roof Panel Weight	3 psf
Floor	Floor Comp	Office/Residential Floor System		
Interior		Residential/Dormitory Interior		

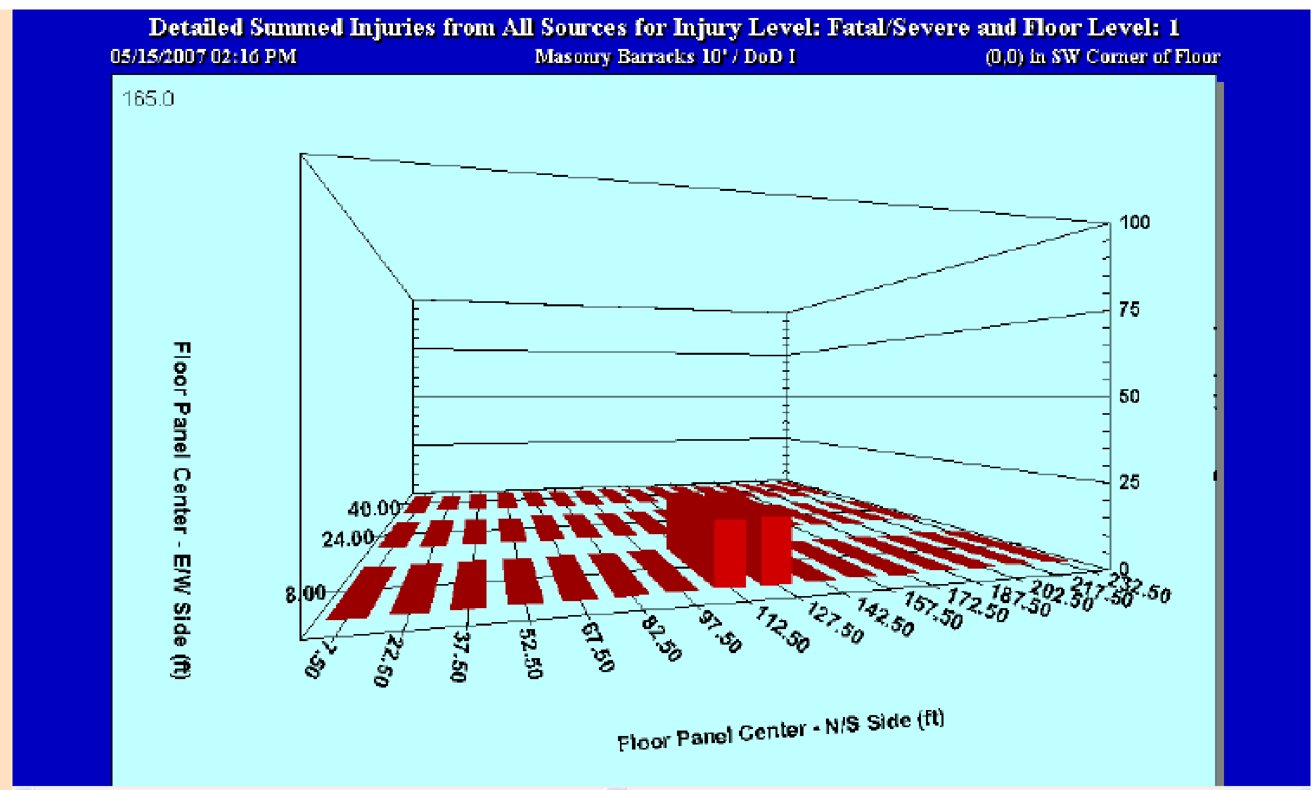
Calculated Blast Loads Summary

	Maximum Load Based on Highest Pressure				Minimum Load Based on Lowest Pressure			
Side	Peak Pressure (psi)	Impulse (psi-ms)	Net* Impulse (psi-ms)	Floor	Peak Pressure (psi)	Impulse (psi-ms)	Net* Impulse (psi-ms)	Floor
North Wall	1.45	28.86	28.86	1	1.45	28.86	28.86	3
East Wall	2.69	24.31	24.31	1	1.93	19.33	19.33	3
South Wall	375.72	329.90	329.90	1	4.50	31.25	31.25	3
West Wall	2.69	24.31	24.31	1	1.93	19.33	19.33	3
Roof	15.62	60.55	60.55	0	2.44	22.79	22.79	0
Floor	0.00	0.00	0.00	0	0.00	0.00	0.00	3

*Net impulse, equal to applied exterior impulse minus room fill pressure impulse, used to get damage from wall and roof components causing injuries. See report for more information on net impulse calculation assumptions.

Total Building Injury Summary**

	Percentage of Injured Occupants					Number of Injured Occupants				
Area	% Fatal/Severe Injury	% Serious Life-Threatening Injury	% Serious Non-Life Threatening Injury	% Minor to Moderate Injury	No Calculated Injury	Fatal/Severe Injury	Serious Life-Threatening Injury	Serious Non-Life Threatening Injury	Minor to Moderate Injury	No Calculated Injury
Total Building	0	0	0	1	99	0	0	0	0	66
Floor 1	1	0	1	2	96	0	0	0	0	21.78
Floor 2	0	0	0	0	100	0	0	0	0	21.78
Floor 3	0	0	0	0	100	0	0	0	0	21.78



Appendix K - Roof Data

Roof Summary:

1. Intent:
    a. In a previous study, charge weight standoff charts were created for a range of various roof systems. The roof components are very unlikely to control the conventional construction standoff. The following charts show that for roof constructions the recommended conventional construction standoff distances would provide an acceptable damage level. Since the roofs pose no serious damage, they can be ignored and the recommendations based solely on wall construction.
2. Components:
    a. Concrete Slab, spanning 6’
    b. Metal Joists
3. Reinforcement Ratios
    a. Lightly = 0.15%
    b. Moderately = 0.25%
    c. Heavily = 0.5%
4. Concrete Strength
    a. f`c = 3,000 psi
5. Roof Systems:
    a. R1 – 300 mm (12 in) heavily reinforced concrete
    b. R2 –300 mm (12 in) moderately reinforced concrete
    c. R3 –300 mm (12 in) lightly reinforced concrete
    d. R4 –225 mm (9 in) heavily reinforced concrete
    e. R5 –225 mm (9 in) moderately reinforced concrete
    f. R6 –225 mm (9 in) lightly reinforced concrete
    g. R7 –150 mm (6 in) heavily reinforced concrete
    h. R8 –150 mm (6 in) moderately reinforced concrete
    i. R9 –150 mm (6 in) lightly reinforced concrete
    j. R10 –100 mm (4 in) heavily reinforced concrete
    k. R11 –100 mm (4 in) moderately reinforced concrete
    l. R12 –100 mm (4 in) lightly reinforced concrete
    m. R13 –18LH08 L=30’ (9.1 m);B=8’ (2.4 m) with metal deck and 5.5” (150 mm) concrete
    n. R14 –18LH05 L=30’ (9.1 m);B=6’ (1.8 m) with metal deck and 4.5” (115 mm) concrete
    o. R15 –18LH02 L=30’ (9.1 m);B=4’ (1.2 m) with metal deck and 3.5” (90 mm) concrete
    p. R16 –30K12 L=30’ (9.1 m);B=8’ (2.4 m) with metal deck and 5.5” (150 mm) concrete
    q. R17 –16K7 L=30’ (9.1 m);B=4’ (1.2 m) with metal deck and 3.5” (90 mm) concrete
    r. R18 –20K10 L=30’ (9.1 m);B=6’ (1.8 m) with metal deck and 4.5” (115 mm) concrete

s. R19 –18LH02 L=30’ (9.1 m);B=4’ (1.2 m) with metal deck
t. R20 –18LH02 L=30’ (9.1 m);B=6’ (1.8 m) with metal deck
u. R21 –16K2 L=30’ (9.1 m);B=4’ (1.2 m) with metal deck
v. R22 –18LH02 L=30’ (9.1 m);B=8’ (2.4 m) with metal deck
w. R23 –16K5 L=30’ (9.1 m);B=6’ (1.8 m) with metal deck
x. R24 –16K9 L=30’ (9.1 m);B=8’ (2.4 m) with metal deck

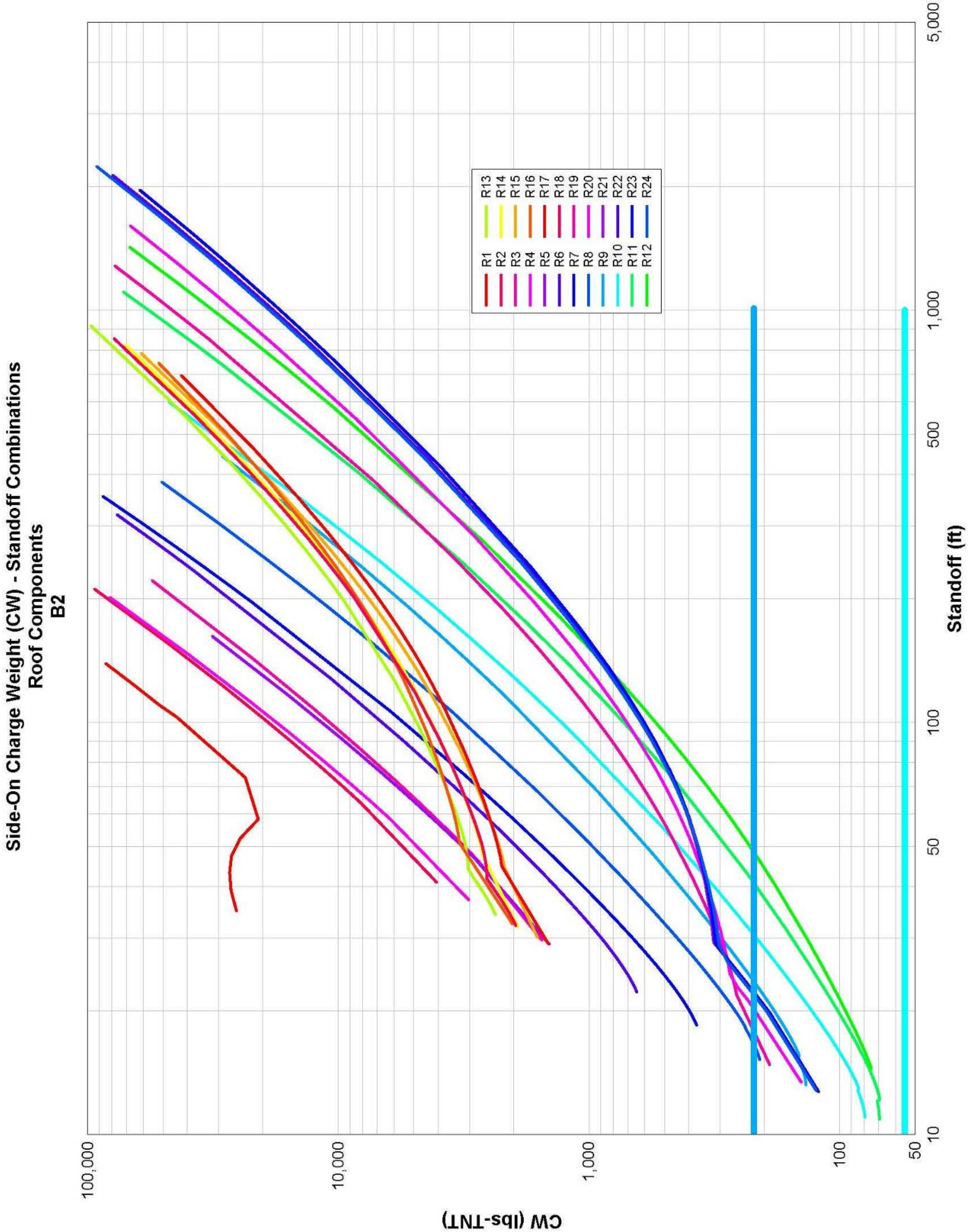


Figure K-1 – B2 Damage Curves for Roofs

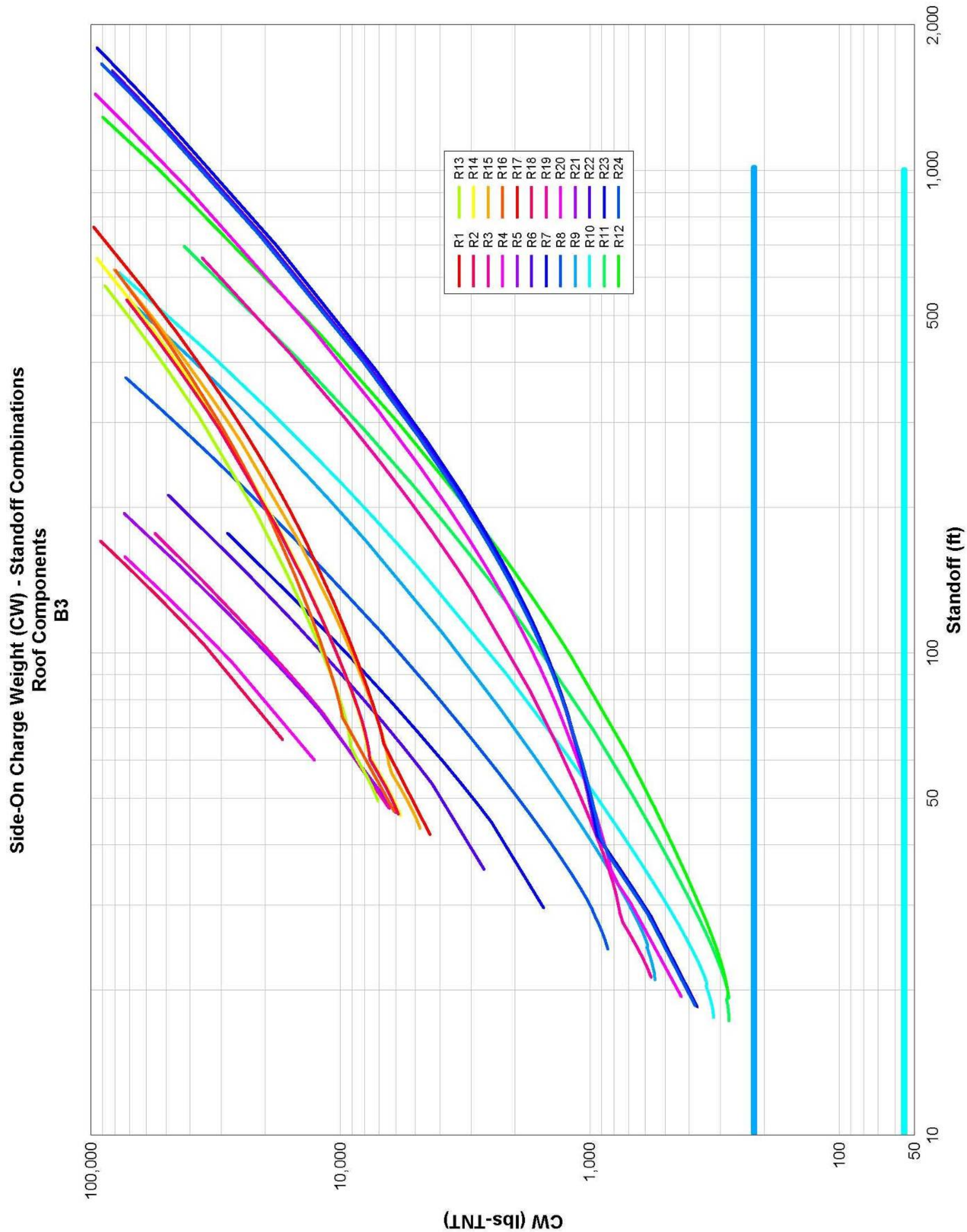


Figure K-2 – B3 Damage Curves for Roofs

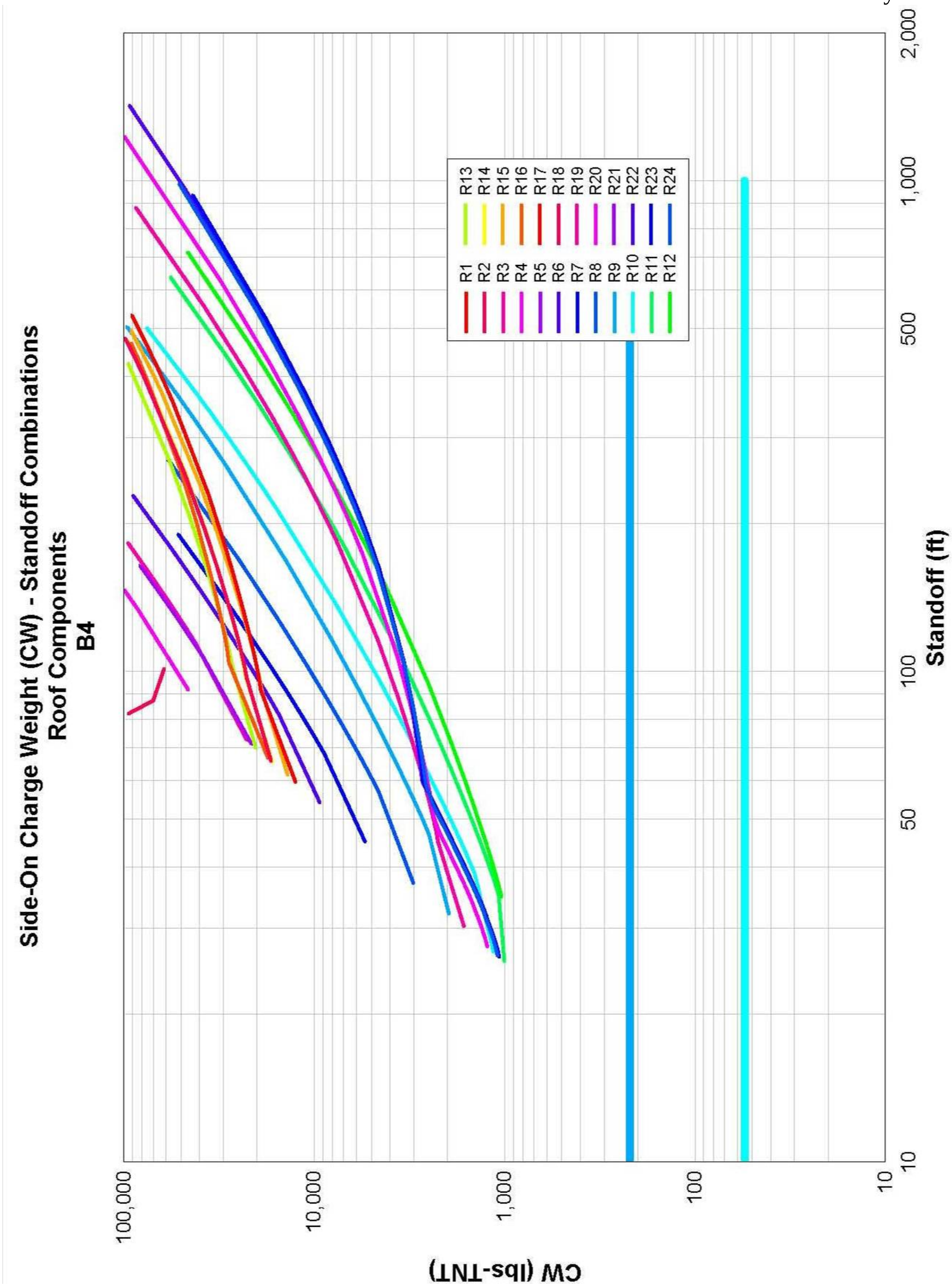


Figure K-3 – B4 Damage Curves for Roofs